THURBER ENGINEERING LTD.

FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF BRIDGE STRUCTURE No. 46-015
SIDEBURNED LAKE BRIDGE HIGHWAY 101
CHAPLEAU TOWNSHIP
G.W.P. 5144-10-00
5015-E-0027

GEOCRES NUMBER: 41O-26

SUBMITTED TO

MCINTOSH PERRY CONSULTING ENGINEERS

Location:
Latitude: 47.7764
Longitude: -83.4896

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual data obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the proposed replacement of the Sideburned Lake Bridge located on Highway 101, within Chapleau Township. Thurber carried out the investigation as a subconsultant to McIntosh Perry Consulting Engineers (MPCE) as part of Agreement No. 5015-E-0027.

No previous foundation investigation information for the subject site was available within the online Geocres Library. However, a historical General Arrangement drawing from 1957 was available and a copy is provided in Appendix A. A Preliminary General Arrangement (GA) drawings and base plan mapping were provided by MPCE for the preparation of this report.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on this data, provide a borehole location plan, record of boreholes, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions.

2 SITE DESCRIPTION

Structure No. 46-015 is located on Highway 101, approximately 8.5 km west of the junction of Highway 129 south of Chapleau, Ontario. It is noted that for project orientation purposes, Highway 101 within the project limits, will be described with an east-west alignment. The location of the bridge is shown on the inset Key Plan on Drawing No. 1 in Appendix A.

Within the project limits, Highway 101 is a two-lane, rural, arterial, undivided highway. Based on the December 2016 drawing the roadway cross-section consists of two, 3.5 m wide lanes, and paved shoulders with a width of 2.0 m and 2.6 m on the WBL and EBL respectively. Steel cable guide rails are located on both sides of the highway for a short distance from the bridge. The southeast steel guide rail is extended with a 3-cabled guide rail.

The existing bridge is an 18.75 m single span, rolled “I” beam bridge with a reinforced cast-in-place concrete deck and was constructed in 1958, see Historic General Arrangement Drawing in Appendix A. The bridge deck was to be horizontal on profile with an elevation of 1460.76 ft (445.2 m).

The bridge deck is approximately 3 m above the water level. The embankment slopes located adjacent to the abutment are inclined at approximately 1.5H:1V with the surface consisting mainly of rock fill with granular infill material. The east approach embankment is built into the water approximately 50 m from the shoreline. Based on the Preliminary GA drawing, the elevation of the center line of roadway is to be approximately 445.095 m and 445.250 m at the east and west abutments, respectively.

Directly adjacent to the south side of the bridge alignment are remnants of a staging platform used to support a bailey bridge for a temporary detour during the initial construction of the bridge. The topography adjacent to the lake at the site is rolling forested lands with frequent bedrock outcrops. The land in the vicinity of the bridge is uninhabited and undeveloped with the exception of a motel which is present east of the bridge site. Traffic volumes are understood to be 425 AADT (2012)

Site photographs showing the general conditions at the site during the time of the field investigation are presented in Appendix D.

3 SITE INVESTIGATION AND FIELD TESTING

Thurber contacted Ontario One Call in advance of the field investigation to provide utility locate clearances in the vicinity of the boreholes.

The field investigation for this site included advancing seven boreholes drilled from October 30, 2016 to November 4, 2016. The northing, easting and elevation of the boreholes are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A and are summarized in Table 3-1.

Table 3-1: Borehole Summary

Borehole No.	Drilled Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Termination Depth below Existing Ground Surface (m)
16-14	East Approach – westbound lane	5293164.3	343042.0	444.5	5.8
16-14B	East Approach – westbound lane	5293161.4	343046.1	444.5	1.9
16-15	East Abutment – westbound lane	5293174.6	343031.7	444.4	19.1
16-16	East Abutment – eastbound lane	5293168.2	343026.6	444.5	16.2
16-17	West Abutment – westbound lane	5293191.1	343006.7	444.7	7.2
16-18	West Abutment – westbound lane	5293186.5	343003.5	444.7	8.7
16-19	West Approach – eastbound lane	5293196.0	342992.2	444.7	6.5

All boreholes were advanced through the roadway embankment with a truck mounted CME 75 drill rig equipped with hollow stem augers and HW/NW casing. The subsurface stratigraphy encountered in the boreholes was recorded in the field by Thurber personnel. Where possible split spoon samples were collected at regular depth intervals in the boreholes via the completion of Standard Penetration Tests (SPT), following the methods described in ASTM Standard D1586. All soil samples recovered from the boreholes were transported to Thurber's Ottawa geotechnical laboratory for further examination and testing.

A 19 mm inside diameter PVC standpipe piezometer was installed in Borehole 16-15 to allow for measurement of the groundwater level at the east abutment following completion of drilling. The piezometer construction details are illustrated on the Record of Borehole sheet for Borehole 16-15, provided in Appendix B. The piezometer was decommissioned November 4, 2016 following completion of the field investigation program.

The boreholes without a piezometer were backfilled with a low-permeability mixture of auger cuttings and bentonite pellets in accordance with Ontario MOE Regulation 903. Boreholes advanced within paved areas were capped with auger cuttings followed by 150 mm of cold patch asphalt to reinstate the travelling surface.

The as-drilled locations and ground surface elevation of the boreholes were surveyed by MPCE in November 2016.

3.1 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples in accordance with the current MTO standards. Grain size distribution analyses testing was also carried out on selected samples to MTO and ASTM standards. Chemical analysis for determination of pH, resistivity, soluble sulphate and chloride concentrations was carried out on two soil samples.

The results of the geotechnical tests are summarized on the Record of Borehole sheets included in Appendix B and all laboratory results are presented on the figures included in Appendix C.

4 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Overview / General

Reference is made to the Record of Borehole sheets in Appendix B for details of the soil stratigraphy encountered in the boreholes. Stratigraphic profiles for the bridge area are presented on Drawing No. 1 in Appendix A for illustrative purposes. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the Record of Boreholes governs any interpretation of the site conditions.

The stratigraphy in the area of the boreholes through the embankment is generally characterized by the asphalt pavement structure and rockfill embankment overlying silty sand or silt above bedrock.

4.2 Asphalt

All boreholes were advanced through the Highway 101 pavement structure. The thickness of the asphalt ranged from 130 mm to 210 mm.

4.3 Fill

Granular Fill

Granular fill consisting predominantly of sand with silt and gravel to gravel with silt and sand was encountered below the asphalt in all boreholes. This layer has a thickness ranging from 0.5 m to 2.7 m (bottom elevation of 441.8 m to 443.8 m). The SPT 'N' values ranged from 41 blows to greater than 100 blows per 175 mm of penetration; indicating a dense to very dense condition.

The moisture content of the samples tested ranged from 1% to 8%. The results of grain size analyses conducted on five samples of this material are summarized in Table 4-1 and are illustrated on Figure C1 in Appendix C.

Table 4-1: Gradation Results for Granular Fill

Soil Particle	%
Gravel	23 to 62
Sand	30 to 66
Silt and Clay	7 to 12

Rockfill

A fill layer consisting predominantly of rockfill was encountered beneath the granular fill in all abutment boreholes (16-15, 16-16, 16-17 and 16-18) as well as in approach borehole 16-14. The voids between rockfill pieces contained a granular infill material. Borehole 16-14 was terminated within this layer at a depth of 5.8 m below ground surface. This layer has a top elevation of 442.9 m to 443.8 m, and a thickness ranging from 2.3 m to 9.9 m where fully penetrated. Boreholes were advanced through the rockfill using casing and coring techniques. Sampling was attempted, however due to the nature of this material sample recovery was poor or not feasible. The SPT 'N' values varied from 5 blows to greater than 100 blows for 200 mm of penetration; indicating a loose to very dense condition. The lower N-values were obtained within the granular infill

Rockfill pieces were cored and indicated particles with diameters ranging from 200 mm to 900 mm. Boulders estimated as large as 1.5 m in diameter were observed on the side slopes of the embankment in the area of the bridge.

4.4 Silt (ML) to Sandy Silt (ML)

A native layer of silt to sandy silt was encountered in Boreholes 16-15 and 16-16. Cobbles and boulders were observed in this unit in both boreholes. The surface of this deposit ranged in elevation from 433.9 m to 434.7 m. This layer has a thickness ranging from 1.7 m to 5.3 m. The SPT 'N' values ranged from 3 to 29 blows per 0.3 m of penetration; indicating a very loose to compact condition.

The moisture content for the samples tested typically ranged from was 13% to 27% with a single moisture content value as high as of 74% recorded near the surface of the layer in Borehole 16-15. The results of grain size analyses conducted on three samples of this material are summarized in Table 4-2 and are illustrated on Figure C2 in Appendix C.

Table 4-2: Gradation Results for Silt (ML) to Sandy Silt (ML)

Soil Particle	%
Gravel	0 to 10
Sand	1 to 24
Silt	58 to 82
Clay	8 to 19

Atterberg Limit testing was completed on one sample of the silt deposit. The result is summarized on the Record of Borehole sheets in Appendix B and the Atterberg Limit graph is included in Figure C3 of Appendix C. The laboratory results indicate that the silt exhibits low plasticity (ML).

4.5 Silty Sand (SM)

A native layer of silty sand with gravel was encountered in Boreholes 16-15, 16-17 and 16-18. Frequent boulders were noted in Borehole 16-15 and wood pieces were present within Borehole 16-17. The surface of this deposit ranged in elevation from 432.2 m to 440.6 m. This layer has a thickness ranging from 0.5 m to 3.0 m. The SPT ‘N’ values ranged from 36 blows to greater than 100 blows per 200 mm of penetration; indicating a dense to very dense condition.

The moisture content for the samples tested ranged from was 10% to 33%.

4.6 Bedrock

The overburden materials were underlain by granite bedrock. Boreholes 16-15 through 16-19 were advanced into the bedrock by coring. The bedrock surface ranges from elevation 429.2 to 441.8 m as summarized in the table below:

Table 4-3 Summary of Bedrock Elevation

Location	Borehole No.	Depth Below Existing Ground Surface (m)	Top of Bedrock Elevation (m)
East Approach	16-14 16-14B	N/A(*)	N/A(*)
East Abutment	16-15	15.2	429.2
	16-16	15.1	429.3
West Abutment	16-17	3.7	441.0
	16-18	5.4	439.3
West Approach	16-19	2.9	441.8

Note: (*) not encountered within the depth of investigation

The Total Core Recovery (TCR) ranged from 98 to 100%, the Solid Core Recovery (SCR) ranged from 95 to 100% and the Rock Quality Designation (RQD) ranged from 72 to 100%. Based on the RQD value the bedrock is classified as fair to excellent quality. Point load strength correlations indicated a strength of very strong or better and Unconfined Compressive Strength tests indicated a compressive strength of 65 to 206 MPa, please refer to Appendix C for UCS test results and rock core photos.

4.7 Groundwater

Groundwater was measured in the open Boreholes 16-14, 16-15, 16-16, and 16-18 during drilling and were noted to range from elevation 441.0 to 441.3m.

The groundwater level was measured in the piezometer installed in Borehole 16-15 on November 4, 2016 at a depth of 3.1 m; corresponding to an elevation of 441.3 m. The water level in Sideburned Lake was measured at the time of Thurber's field investigation at an elevation of 441.3 m.

These observations are considered short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy and/or prolonged precipitation. Due to the open nature of the rockfill approach embankments, it is expected that the groundwater level will respond rapidly to the water level changes in Sideburned Lake.

5 MISCELLANEOUS

Thurber obtained utility clearances prior to drilling and the borehole locations were positioned relative to existing site features and proposed foundations. MPCE surveyed the borehole locations and ground surface elevations. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario supplied and operated the drilling equipment to carry out the drilling, sampling, in-situ testing, standpipe piezometer installation and borehole decommissioning. The drilling, and sampling operations in the field were supervised on a full- time basis by Mr. Christopher Murray, P.Eng. of Thurber. Laboratory testing was carried out in Thurber's MTO-approved laboratory in Ottawa.

Overall project management and direction of the field program was provided by Mr. Stephen Peters, P.Eng. Interpretation of the field data and preparation of this report was completed by Mr. Justin Gray P.Eng. The report was reviewed by Dr. Fred Griffiths, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This section of the report presents interpretation of the factual data in Part 1 of this report for the proposed replacement of the Sideburned Lake Bridge located on Highway 101, near Chapleau, Ontario. Geotechnical assessment and recommendations are provided to assist the project team in designing a suitable foundation for the proposed replacement bridge.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The construction or design-build contractor must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The existing 18.75 m long by 13.4 m wide bridge is supported by differing foundation types. The existing west bridge abutment is supported on two caissons bearing on the bedrock surface near approximate elevation 436.8 m (as noted on Historical GA drawing dated June 1957 see Appendix A). Each caisson is laterally supported with a single deadman anchor buried within the approach fill. The existing east bridge abutment is supported on a spread footing above a tremie concrete base slab perched within the rockfill.

Settlement has been noted to have occurred at the approach within the 2015 Ontario Structure Inspection Manual. As noted in Section 2 above, the Historic GA (copy provided in Appendix A) indicates that the bridge deck was to be constructed with a flat profile at elevation 1460.76 ft (445.2 m). The current ground surface elevation near the east abutment at Boreholes 16-15 and 16-16 is approximately 444.4 m to 444.5 m and near the west abutment it is 444.7 m at both Boreholes 16-17 and 16-18. Given the presence of shallow rock and rockfill, it is unlikely that the west approach settled 0.5 m thus there may be a conversion discrepancy from historic to current elevations. Nonetheless it is apparent that the east approach is now 0.2 to 0.3 m lower than the west approach while the Historic GA indicates they should have been be at the same level when constructed. This settlement can be attributed to reorientation of rockfill over time that had initially been placed loosely by dumping under water.

No previous foundation investigation information for the subject bridge was available within the online Geocres Library.

The following sections address the foundation aspects of the installation of the new bridge. The discussions and recommendations presented in this report are based on the information provided by MPCE including the 30% Contract Drawings dated September 2017 and on the factual data obtained during the course of the investigation.

6.1 Proposed Structure

At the time of preparation of the Foundation Investigation and Design Report, the design of the proposed bridge structure is shown on Sheet 59 of the Contract Drawings to consist of a 10.5 m wide by 31.0 m long single span bridge consisting of 6 NU1200 concrete girders. The longer bridge will be placed along the same alignment as the existing bridge and the west abutment (elev. 442.25 m) is indicated to be founded directly on bedrock and the east abutment (elev. 442.25 m) is indicated to be founded on a widened tremie pad bearing on the existing rock fill. The new bridge abutments will be constructed behind the existing abutments.

Due to the existing pair of caissons and deadman anchors at the west abutment, it has been determined in the Technically Preferred Alternative (TPA) memorandum that it is not structurally feasible to replace the existing structure while also maintaining traffic on the existing alignment. Therefore, it has been proposed to divert traffic to the south of the current bridge alignment with the use of a temporary, single lane modular bridge supported on staging platforms currently understood to be remaining in place from the initial bridge construction. The foundation conditions for the temporary detour alignment were not investigated as part of this assignment and a separate field investigation for the foundation for the temporary detour and modular bridge along the south side of the highway alignment has been undertaken and recommendations are proved within a separate report (Geocres 41O-27).

6.2 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6-14.

The frost penetration depth at this site is 2.4 m as per OPSD 3090.100.

In accordance with CHBDC CSA S6-14, the analysis and design of structures takes into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which in this case is the Ministry of Transportation, Ontario (MTO).

It is understood that MTO has designated this structure as follows:

Table 6-2: Bridge Structure Classification

Criteria	Classification	CHBDC Section
Importance Category	Major Route Bridge	4.4.2
Consequence Classification	Typical Consequence	6.5.1

Based on the above, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances.

7 SEISMIC CONSIDERATIONS

7.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data includes peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values (Sa(T)) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is presented in Appendix F.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA). The PGA value at this site for a *reference* Site Class C with a 2% probability of exceedance in 50 years (2475-year event) is 0.040g. This value is to be scaled by the site-specific Site Class as discussed below

7.2 CHBDC Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic site classification is based on the least favourable soil conditions encountered in the upper 30 m of the stratigraphy.

Based on the soil and bedrock conditions encountered below the anticipated bridge foundation elevation, the site is classified as a Seismic Site Class C in accordance with Table 4.1 of the CHBDC.

7.3 Seismic Liquefaction

Based on the subsurface conditions encountered at the drilled locations at this site, the foundation soils are considered to be not susceptible to liquefaction during a seismic event taking into consideration the low PGA values.

8 STRUCTURE FOUNDATION ALTERNATIVES

Given the soil stratigraphy encountered, the following options have been considered for the new bridge foundations:

- Spread footings perched within rockfill or bearing on bedrock
- Caissons socketed into bedrock (drilled shafts)
- Steel piles (H-piles, pipe piles)

These foundation alternatives are presented in the following sections and evaluated from a geotechnical perspective in terms of their respective advantages, disadvantages, risks and consequences. The evaluation is summarized in the table provided in Appendix E. A preferred replacement alternative from a geotechnical engineering perspective is recommended.

8.1 Spread Footings

The existing east abutment is supported on a spread footing founded on a 1.5 m thick tremie concrete base slab perched within the rockfill approach embankment. Supporting the new west or east bridge abutment on concrete spread footings can be considered feasible at this site. It is understood that structurally, spread footings would need to be placed no higher than 441.3 m at the east abutment. Given the open nature of the rockfill, the groundwater level will be similar to that in Sideburned Lake which was observed to be at elevation 441.3 m at the time of the foundation investigation. Geotechnically, spread footings must be provided with adequate protection against ice jacking. Spread footings will be difficult to construct below the groundwater table and do not allow for the construction of integral abutments, if required. To reduce the dewatering efforts, the base of the footing would have to be elevated above the expected water level. The abutments would also require subgrade preparation including chinking of the rockfill to reduce loss of bedding material into the rockfill. The excavation depth and limits for preparation of the footing subgrade should be reviewed to insure it would not destabilize the adjacent temporary detour footings. Spread footing not founded on bedrock will have a greater potential for settlement compared to deep foundations alternatives and the bridge structure would need to be designed with an appropriate geotechnical capacity at SLS or be designed to tolerate these settlements.

8.2 Caissons

The existing west abutment is supported on a pair of 2.1 m diameter caissons bearing directly on the underlying bedrock but required deadman to provide lateral resistance. Caisson foundations, particularly when they are socketed into bedrock, offer high geotechnical resistance, however, their high lateral stiffness is not compatible with requirements for integral abutments. Caissons typically require less space to install but will encounter difficultly drilling through rockfill due to their expected diameter and due to unbalanced water heads. Permanent liners would be required to keep the drill holes open and to allow dewatering for inspection of the base of the caissons. Caissons are feasible at this site however, it is understood that the nearest ready mix concrete plant is not located near the site and the volume of concrete required for caissons will need to be reviewed.

8.3 Steel Piles

It is understood that the underside of pile caps would need to be placed no higher than 441.1 m at the east abutment for structural purposes. Driven steel H-piles are not recommended at the west abutment due to the shallow depth of bedrock and the resulting short length of pile. Steel H-piles driven to bedrock with a rock point tip are considered feasible at the east abutment. However, pre-drilling will be required to install the piles through the rockfill. In addition, the underlying silty sand deposit contains cobbles and boulders which could also obstruct pile driving.

Installing drilled steel pipe piles is considered feasible and would be more economical than driven H-piles as pre-drilling through the rockfill would be required as part of the H-pile installation. Drilling would also reduce the likelihood of misalignment from driving a pile and the casing would act as a liner to keep the drill holes open.

8.4 Recommended Foundation

Based on the proposed structure geometry and the evaluation of foundation alternatives presented above, drilled steel pipe piles socketed into bedrock are considered a feasible and cost effective option and are recommended at both abutments. However, spread footing with a low geotechnical resistance at SLS are also considered a feasible alternative provided river levels at the time of construction permit excavation to elevation 440.3m at the east abutment.

9 FOUNDATION DESIGN RECOMMENDATIONS

9.1 Steel Pipe Piles

The potential exists for slipping and damaging of the pile tip on a sloping bedrock surface if driving below the rockfill to refusal, therefore it is recommended that steel pipe piles should be drilled-in full depth and socketed into the bedrock. For pipe piles socketed 1.0 m or more below the bedrock surface with a sidewall thickness of 12.7 mm or greater, factored geotechnical resistances at ULS for an end bearing pile are provided in the table below. A resistance factor (ϕ_{gu}) of 0.4 has been included in the ULS values as per Table 6.2 of the CHBDC (static analysis – typical understanding). The SLS condition will not govern for piles socketed into bedrock. It is understood that equipment to install drilled-in pipe piles with diameters of 508 mm (20 inches) or less are more readily available with local contractors and the recommendations have been provided as such.

Table 9-1 Recommended Resistance Values for Drilled-In Steel Pipe Piles

Pile Diameter (mm)	Factored Resistance at ULS (kN)
324	2,000
406	3,000
508	5,000

The resistance values presented above have been reduced to account for the possibility that residual crushed rock may remain as the base of the drilled-in pile. The depth of socket into bedrock may need to be greater than 1.0 m to address the lateral resistance requirement, base fixity requirement and shear and moment demand for each pile.

The geotechnical resistance values assume a minimum centre-to-centre spacing of three pile diameters. The resistance values will need to be reduced for lesser pile spacing.

The method of installation of drilled-in pipe piles is the responsibility of the Contractor. It is expected that pile installation will encounter cobbles and boulders in the native soils underlying the rockfill. The Contractor’s drilling equipment should be capable of dislodging, handling and removing these obstructions. Care must be exercised while drilling into bedrock. The drilling methodology must be capable of advancing the pile without disturbing or fracturing the bedrock at the base of the pile. Blasting to facilitate rock removal is not permitted. The bedrock is expected to be hard. The drilling equipment selected by the contractor must be capable of advancing into the bedrock.

Since the rock cutting shoe at the tip of the pipe pile will be slightly larger in diameter than the outside diameter of the pile, there will be a small gap between the rock wall and the pile. It is recommended that the annular space between the pile and rock wall be grouted to the bedrock surface to achieve fixity. The pipe piles may be partially filled with water and tremie concreting will be required for concreting these pipe piles.

The base of all buried pile caps must be provided with earth cover or thermal equivalent as protection against frost action (Section 9.5). The soils in front of the piles should be protected from scour so that the piles do not lose lateral support.

9.1.1 Downdrag

Downdrag on the piles is not considered to be an issue at this site since with the proposed grade raise of less than 0.7m and the low clay content in the foundation soils. It is recommended that the approach fills for the temporary detour be built in advance of pile installation to limit any settlement occurring after installation.

9.1.2 Lateral Resistance of Piles

Resistance to lateral movement of a pile foundation will be provided by the passive earth pressure developed on the face of the pile embedded in the non-cohesive foundation soils and bedrock.

The geotechnical lateral resistance that can be mobilized in front of the pile in the overburden may be analysed using a soil-spring model and computed using the coefficient of horizontal subgrade reaction k_s and ultimate lateral resistance p_{ult}. The value of k_s varies with depth and may be calculated as follows:

$$k_s = n_h * z / D$$

$$p_{ult} = 3 * \gamma' * z * K_p$$

where:

n_h = coefficient related to soil density, see table below (kN/m^3)
z = depth of embedment of pile (m)
D = pile diameter (m)
γ' = effective unit weight of soil, see table below (kN/m^3)
K_p = passive earth pressure coefficient, see table below (-)

The parameters recommended for the use with the above equations are provided in Table 9-2.The ultimate passive resistance force that can be mobilized by the embedded portion of a socket within bedrock is constant with depth and is given by:

$$P_p = 6 * c * z * L$$

where:

c = 2,000 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)
z = depth of embedment of pile (m)
D = pile diameter (m)

Table 9-2 Parameters for Lateral Pile Resistance

Location	Elevation (m)	Unit Weight(*) (kN/m^3)	n_h (kPa/M)	K_p (-)	Soil
West Abutment (Borehole 16-18)	442.0 to 441.3	18	20,000	4.6	Rockfill (above water)
	441.3 to 440.6	8	10,000	4.6	Rockfill
	440.6 to 439.3	10	5,000	3.3	Silty Sand
	439.3 to Base	-	-	-	Bedrock
East Abutment (Borehole 16-16)	441.8 to 441.3	18	20,000	4.6	Rockfill (above water)
	441.3 to 434.7	8	10,000	4.6	Rockfill
	434.7 to 429.3	9	5,000	3.3	Silt
	429.3 to Base	-	-	-	Bedrock

Note: (*) Submerged unit weights have been provided below the water table

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The factored lateral resistance of the piles determined based on the data and methods provided above should incorporate a resistance factor (ϕ_{gu}) of 0.5 as per Table 6.2 of the CHBDC (static analysis – typical understanding). The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K_s, for analysis may be obtained by the expression, $K_s = k_s * L * D$ (kN/m), where L is the length (m) of the pile segment or element used in the analysis and the remaining parameters are as defined earlier. The ultimate lateral resistance, P_{ult}, on any one segment of pile may be obtained from the expression, $P_{ult} = p_{ult} * L * D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. However, it is recommended that the total lateral resistance for one pile be limited to no more than 100 kN at ULS and 45 kN as SLS.

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile center-to-center spacing less than 4 pile diameters. The factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Figures C6.11.2(r), C6.11.3(s) and C6.11.3(t) of the CHBDC.

9.2 Spread Footings

The geotechnical bearing resistances provided in this report for spread footings include a resistance factor of 0.5 (ϕ_{gu}) and 0.8 (ϕ_{gs}) for the ULS and SLS values, respectively, as per Table 6.2 of the CHBDC (static analysis – typical understanding). The geotechnical resistances presented in the following subsections are for vertical concentric loading only and will need to be adjusted for the effects of inclined or eccentric loadings, where applicable, in accordance with CHBDC Clause 6.10.3 and 6.10.4.

9.2.1 Spread Footing on Bedrock at the West Abutment

The depth to bedrock in the boreholes advanced at the west abutment was noted to range from 2.9 to 5.4 m below the existing road grade. The existing overburden should be excavated and the spread footing should be founded directly on the bedrock. The lowest elevation of bedrock observed in the boreholes was 439.3 m which is 2.0 m below the water level noted during the time of the field investigation. Where bedrock is exposed it should be inspected and excavated to create a horizontal surface or alternatively, the founding elevation can be raised with the use of a concrete plug in accordance with OPSS.PROV 904 with the same class of concrete as the footing to reduce the excavation and dewatering efforts.

A spread footing at the west abutment founded on the bedrock can be designed with a factored geotechnical resistance at ULS of 1500 kPa. SLS will not govern design for a footing founded on bedrock.

The horizontal resistance against sliding between a cast-in-place concrete footings founded on bedrock can be computed using a friction factor of 0.70. Appropriate resistance factors should be applied for the design. Alternatively, anchors or shear pins could be used to provide additional capacity.

9.2.2 Spread Footing on Rockfill at the East Abutment

At the east abutment, a spread footing can be constructed on an engineered pad consisting of 1.0 m thick Granular ‘A’ material placed on a geotextile over the existing rockfill. Alternatively, the founding elevation can be raised with the use of a tremie concrete plug to reduce the dewatering efforts. The engineered pad can bear on the exposed rockfill subgrade provided it is free of any soft or deleterious materials and the surface of the rockfill is chinked. For additional protection, a geotextile (Class II non-woven FOS 50 to 150 μm, OPSS 1860) should be placed as a separator between the Granular ‘A’ and the rockfill. The top of the Granular ‘A’ pad must extend to 1.0 m beyond the surface of the edge of all sides of the footing and be sloped away from the footing at 1H:1V, or flatter. The founding elevation of the base of the footing should take into consideration the elevation of the water and the potential for ice jacking. It is recommended that the existing tremie concrete pad be left in place and not removed. However, portions of the tremie pad may need to be removed as part of the subgrade preparation for the new footing. If left in place, old and new tremie pads may not settle the same amount under the new footing creating an abrupt differential settlement across the footing. This is mitigated by the low amount of settlement anticipated for the SLS value recommended below.

The following factored geotechnical resistance values are recommended for a 3.0 m wide cast-in-place footing positioned behind the existing tremie pad and founded on a 1.0 m thick engineered fill pad at this site. The geotechnical resistances provided take into account the proximity to the steep forward slope.

- Factored Geotechnical Resistance at ULS of 300 kPa
- Factored Geotechnical Resistance at SLS of 170 kPa

The rock fill beneath the footing will have minimal settlement (less than 10 mm), thus differential settlement between the east and west abutments will also be limited. Differential settlement across the east footing will also occur from west to east due to the past loading from the previous foundation, however, this will be less than 10 mm.

Implicit in the design for the east abutment is the requirement that the approach fills not be raised more then the maximum on 0.7 m as is currently being proposed. Fills placed for the temporary detour will need to be removed upon completion of the new structure.

The horizontal resistance against sliding between a cast-in-place concrete footing founded on engineered fill can be computed using a friction factor of 0.55. Appropriate resistance factors should be applied for the design.

Resistance to lateral forces/sliding resistance between the foundations and the underlying subgrade should be calculated using an ultimate coefficient of friction of 0.45. Appropriate resistance factors should be applied for the design.

9.2.3 Wingwalls

Wingwalls perched in the rock fill approaches at this site should be founded on a leveling pad consisting of Granular 'A' material with a minimum thickness of 0.5 m. The engineered pad should be placed on a geotextile separation (Class II non-woven FOS 50 to 150 μm) and can bear on the existing approach fills provided it is free of any soft or other deleterious materials and the surface of the rockfill is chinked. The top of the Granular 'A' pad must extend to 0.5 m beyond the outside edge of all sides of the footing and sloped at 1H:1V, or flatter.

The following factored geotechnical resistance values are recommended for design of wingwall foundations as wide as 1.5 m at this site:

- Factored Geotechnical Resistance at ULS of 450 kPa
- Factored Geotechnical Resistance at SLS of 300 kPa

Considering the competency of the foundation soils, settlement of the foundation soils under the loading imposed by the wingwalls is expected to be negligible.

9.3 Rock Anchors / Dowels

It is understood that vertical rock anchors and/or dowels will be utilized at the west abutment. Rock anchors/dowels grouted into the underlying bedrock are considered to be feasible at this site to provide additional vertical resistance. However, the additional vertical loading from the pre-tensioned rock anchors will need to be incorporated into the design of forces acting on the foundation. All overburden must be removed from above the bedrock surface. Resistance from weathered/fractured bedrock should be ignored and not included in the calculation of available anchor/dowel capacity. Based on a minimum grout strength of 30 MPa, a rock anchor or dowel installed within sound bedrock can be designed with an ultimate bond stress of 1000 kPa. A geotechnical resistance factor of 0.4 (ϕ_{gu}) as per Table 6.2 of the CHBDC (static analysis – typical understanding) is to be applied to the calculated value. The lower of the grout to anchor/dowel bond and grout to bedrock bond should be used in design. A minimum rock anchor length of 3 m into sound bedrock and a minimum dowel length of 1.5 m into sound bedrock should be used in design irrespective of the calculated capacity. Rock anchor design, installation and proof testing should be in conformation with OPSS 942. An NSSP on the supply, installation and testing of rock dowels is provided in Appendix F. Rock anchors/dowels should be provided with double corrosion protection.

The Contractor's drilling equipment must be able to penetrate in to the sound bedrock to achieve the design bond length. When installing the rock anchors/dowels, the pre-drilled holes shall be free of dust and debris prior to placement of the anchoring agent. The anchors/dowels shall be maintained in position during the setting of the anchoring agent and loss of anchoring agent form the holes shall be prevented.

A check should be completed to verify the calculated bond strength does not exceed the effective unit weight of rock encompassed within an inverted cone inclined at 45 degrees from vertical acting from the base of the bonded length of the anchor/dowel to the surface of the sound rock. Additionally, individual rock anchor/dowel capacity should be reviewed and reduced taking into consideration the proximity of other structural and foundation elements that encroach within the circumference of the inverted cone.

9.4 Subgrade Preparation

Subgrade preparation for the abutment and wingwall (as needed) foundations should include the removal of the existing granular fill and any loose, soft or organic materials within the footprint of the proposed foundation.

The base the excavations should be inspected by qualified geotechnical personnel in accordance with SP109S12 prior to placing the granular pad in order to confirm that the founding conditions are consistent with the recommendations described herein, and to ensure that there is no disturbance of the soil within the abutment and wingwall footprints. Any deleterious materials, organics, or loose/soft or wet conditions observed, should be sub-excavated and removed and the excavations backfilled with OPSS Granular B Type II compacted as per OPSS.PROV 501.

9.5 Frost Protection

The frost penetration depth at this site is 2.4 m as per OPSD 3090.100. Footings founded on sound bedrock or founded on mass concrete which is on sound bedrock, do not require frost protection. For all other footings and pile caps, a minimum of 2.4 m of earth cover, or thermal equivalent, must be provided above the base of the footing and pile cap to serve as protection against frost. Thermally equivalent frost protection could be in the form of polystyrene insulation provided it is placed above the highwater level.

It should be noted that rock fill does not provide equivalent frost protection as earth.

9.6 Backfill and Lateral Earth Pressure

Backfill behind the abutments should be placed in accordance with OPSS 902. All backfill material should consist of Granular A, or Granular B Type II meeting OPSS.PROV 1010 specifications. The backfill must be in accordance with OPSS 902 and placed to the extents shown on OPSD 3101.150.

The backfill should be compacted and compaction equipment to be used adjacent to the walls should be restricted in accordance with OPSS.PROV 501. The design of the abutment and wingwalls must incorporate a subdrain as shown in OPSD 3101.150. If adequate drainage cannot be confirmed, the potential of hydrostatic pressures should be considered.

9.6.1 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but, under fully drained conditions, is generally given by the expression:

$$\sigma_h = K^*(\gamma d + q)$$

where:

σ_h = static horizontal pressure on the wall at depth d(kPa)
K = static earth pressure coefficient
γ = unit weight of retained soil (kN/m^3)
use submerged unit weights below water
d = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC. The recommended lateral earth pressure parameters for use in the design for a vertical structure are provided in Table 9-3.

Table 9-3 Static Lateral Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II ϕ = 35^o, γ -= 22.8 kN/m^3		OPSS Granular B Type I ϕ = 32^o, γ -= 21.2 kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active, K_A (Yielding Wall)	0.27	0.40	0.31	0.48
Active, K_o (Non-Yielding Wall)	0.43	-	0.47	-
Active, K_P (Movement towards soil mass)	3.7	-	3.3	-
Soil Group$^{(*)}$	‘medium dense sand’		“loose to medium dense sand”	

Note: (*) for use with Figure C6.16 of the commentary to the CHBDC

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The parameters in the table correspond to full mobilization of active and passive earth pressure and require certain relative movements between the wall and adjacent soil to produce these conditions. The values used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC using the soil group designate as outlined in the Table. Where ground surfaces are sloped behind the walls, the corresponding coefficients should be used.

For static analysis, passive earth resistance in front of the abutments should be ignored. A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Section 6.12.3 of the CHBDC.

9.6.2 Combined Static and Seismic Lateral Earth Pressure Parameters

Retaining structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. The following recommendations are per Section C4.6.5 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with:

- k_h = ½ F(PGA)•PGA for structures that 25 mm to 50 mm of movement, and
- k_h = F(PGA)•PGA for non-yielding walls

The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002 for a yielding structure with respect to the assessment of seismically induced lateral earth pressures.

The recommended seismic lateral earth pressure parameters for use in the design of vertical walls are provided in Table 9-4. The provided earth pressure coefficients are based on a Seismic Site Class, *reference* PGA with a 2% probability of exceedance in 50years of 0.040g (Geological Survey of Canada – Fifth Generation).

Table 9-4 Dynamic Lateral Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^o$, γ -= 22.8 kN/m^3		OPSS Granular B Type I $\phi = 32^o$, γ -= 21.2 kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active, K_{AE} Yielding Wall	0.28	0.42	0.32	0.51
Active, K_{AE} Non-Yielding Wall	0.30	0.45	0.33	0.54

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K \gamma d + (K_{AE} – K_A) \gamma (H - d)$$

where:

σ_h = lateral earth pressure on wall at depth, d (kPa)
d = depth below the top of the wall where pressure is computed (m)
K = static active earth pressure coefficient
(K_a for yielding walls, K_o for non-yielding walls)
γ = unit weight of the backfill soil (kN/m^3)
use submerged unit weights below water
K_{AE} = combined static and seismic earth pressure coefficient
H = total height of the wall (m)

9.7 Cement Type and Corrosion Potential

Two samples of the native soils were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, resistivity and conductivity. The analysis was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in the Table 9-5.

Table 9-5: Results of Chemical Analysis

Borehole	Sample	Depth (m)	Sulphate (µg/g)	pH	Resistivity (Ohm-cm)	Chloride (µg/g)
16-15	SS6	12.4	54	7.8	5840	24
16-18	SS6	4.9	18	6.8	2850	171

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. The class of concrete selected should consider the effects of road de-icing salts.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in the Table 9-5 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The effects of road de-icing salts should also be considered.

9.8 Embankment Design and Reinstatement

Embankment reconstruction should be carried out in accordance with OPSS.PROV 206. The embankment should be reinstated with side slopes of 2H:1V (or flatter) if constructed using Select Subgrade Material (SSM) or Granular B Type I. To match the existing slope of 1.5H:1V, rockfill should be used.

Where new embankment fill is placed against existing embankment slopes or on a sloping ground surface steeper than 3H:1V, benching of the existing slope should be carried out in accordance with OPSD 208.010.

Provided construction of embankment and cut slopes are carried out in accordance with recommendations provided within this report, the minimum required factor of safety will be maintained for static and seismic loading conditions.

It is understood that a grade raise of 0.55 and 0.70 m is anticipated along Highway 101 at the west and east abutments, respectively. Settlement within the foundation soils is expected to occur. If approach fill is placed prior to foundation construction, negligible settlement is expected to occur after construction The magnitude of compression for an embankment constructed with granular materials or rockfill is in the order of 0.5% of the fill height and is expected to occur following fil placement. For dumped rockfill (placed under the water level), these compression values would be approximately doubled. Placement of the final lift of asphalt should be delayed for at least one month.

9.9 Temporary Detour Structure

The foundation conditions and design recommendations for a temporary detour along the south side of the highway alignment is provided in Geocres 41O-27.

10 CONSTRUCTION CONSIDERATIONS

10.1 Excavations

It is anticipated that temporary excavations greater than 4.5 m are expected for the removal of the existing footings. All excavations must not encroach within 1H:1V from the base of the excavation to the temporary detour bridge support.

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fills and native soils above the water table and the rockfill below the water table at the site should be classified as Type 3 in accordance with OHSA.

Excavation for the structure replacement must be carried out in accordance with OPSS 902. Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. Stockpiling or surface surcharge is also the responsibility of the Contractor and must not compromise temporary or permanent slopes.

At locations where there are space restrictions or where a slope has to be retained, the excavations will need to be carried out within a protection system. Design of the temporary protection system is the responsibility of the Contractor.

10.2 Temporary Protection Systems

It is understood that a full road closure will be utilized during construction and therefore a temporary protection system (TPS) is not anticipated. In addition, the installation of a TPS will be difficult given the existing foundation soils consist of rockfill on the east side and shallow bedrock on the west.

However, if a TPS is required as part of construction activities, the design of the TPS is the responsibility of the Contractor and all TPS's should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. Temporary protection systems should be provided in accordance with OPSS.PROV 539 and designed for Performance Level 2 (maximum 25 mm horizontal deflection). The actual pressure distribution acting on the shoring systems is a function of the construction sequence and relative flexibility of the wall and these factors must be considered when design the shoring system. Thurber can provide geotechnical parameters upon request.

10.3 Dewatering

Subgrade preparation and placement of granular pads and abutments must be carried out in the dry. Maintaining a dry excavation in rockfill below the water table will be difficult. The use of a tremie concrete to form a concrete plug, as indicated in Section 9.2.2, can be utilized to bring the founding elevation above the water and reduce the dewatering concerns.

The Contractor must be prepared to control the groundwater and surface water flow at the site to permit construction in a dry and stable excavation

Water from either surface flow and/or groundwater must be diverted away from the excavation at all times. Groundwater perched within the embankment fill and, surface runoff will tend to seep into, and accumulate in proposed excavations.

Dewatering and surface water diversion must remain operational and effective until the temporary excavation is backfilled. Design of an effective dewatering system must be carried out by the Contractor. Dewatering systems should be designed, operated and removed in accordance with OPSS.PROV 517 and Special Provision No. 517F01 with the following inputs for Table A: Note 1 = Yes and ****** = N/A. The assessment for the need for a Permit to take Water (PTTW) should be carried out by a specialist experienced in this field.

10.4 Erosion Control and Scour Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. The embankment materials primarily consisting of sand and gravel, rockfill and silty sand are all considered to have a low erosion potential. The native silt is considered to have a moderate erosion potential. Slope vegetation should be established as soon as possible after completion of the earth embankment fills in order to control surficial erosion in general accordance with OPSS.PROV 804. The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediment from running off the site as per OPSS 805.

Scour and erosion protection should be provided to protect the integrity of the foundations and the embankments. Design of the scour and erosion protection measures must consider hydrologic and hydraulic concerns and should be carried out by specialists experienced in the field. Typically, rock protection should be provided over all earth surfaces subjected to flowing water in accordance with OPSS 511.

11 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to, the following:

- The existing west abutment is supported on a pair of caissons and lateral resistance is provided with a pair of deadman anchors buried within the approach fill. The stability of the abutment is based on the support from the deadman anchors and appropriate steps should be taken when dismantling the existing bridge. The Contractor should be alerted to these conditions.
- Cobbles, boulders and rockfill or other buried obstructions will be encountered in the existing approach embankments. An NSSP should be included in the contract alerting the Contractor to these conditions.
- The Contractor's selection of construction equipment and methodology should include assessment of the capability of the subgrade soils to support the proposed construction equipment and any temporary structures or fill (i.e. as a pad for crane support).
- Seasonal fluctuations of the groundwater and river level are to be expected which may impact the construction. Dewatering in rockfill will be difficult.

The successful outcome of the project will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations by qualified geotechnical personnel in accordance with SP109S12 will be required during construction to confirm that the foundation recommendations are correctly implemented, and material specifications are met.

12 CLOSURE

Engineering analysis and preparation of this report was completed Stephen Peters, P.Eng. The report was reviewed by Dr. Fred Griffiths, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.

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APPENDIX A

BOREHOLE LOCATIONS AND SOIL STRATA DRAWINGS

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

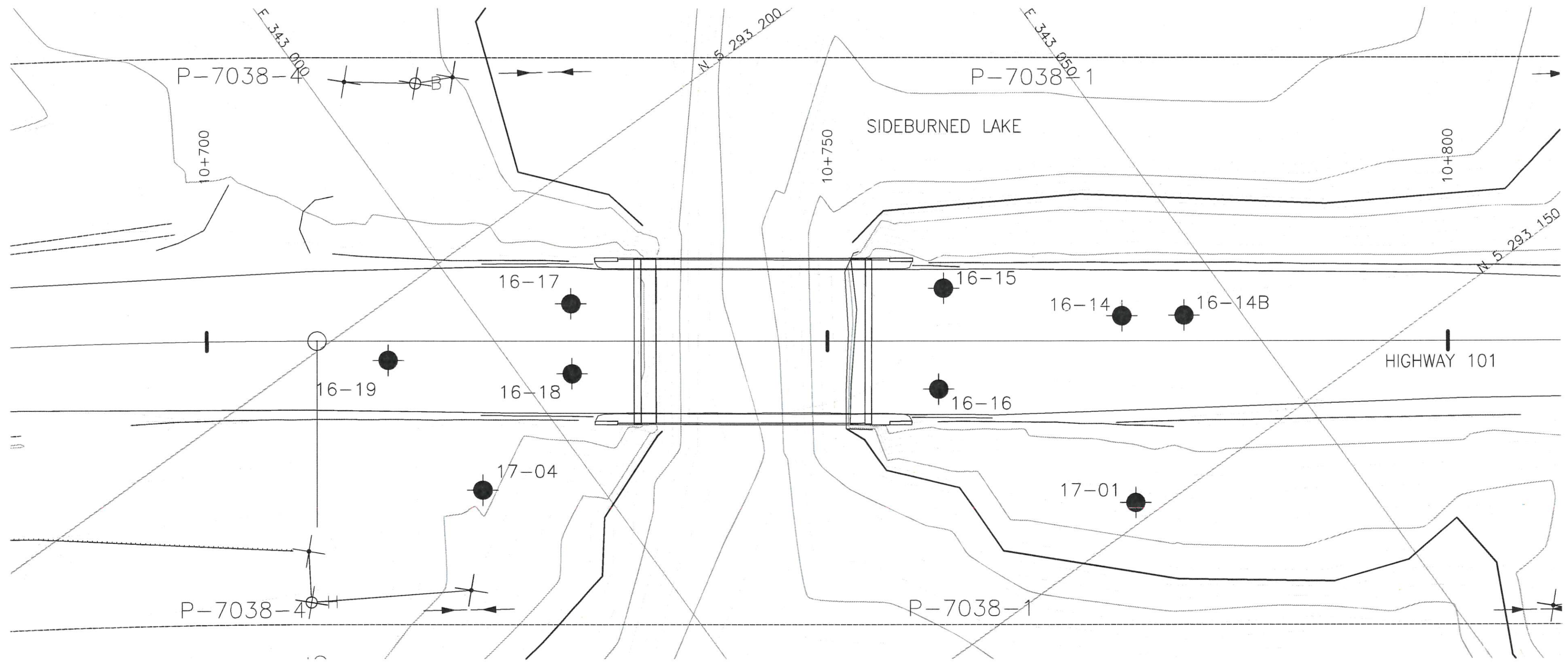
CONT No
GWP No 5144-10-00

HIGHWAY 101
SIDEBURNED LAKE
BRIDGE REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

McINTOSH PERRY

THURBER ENGINEERING LTD.



PLAN

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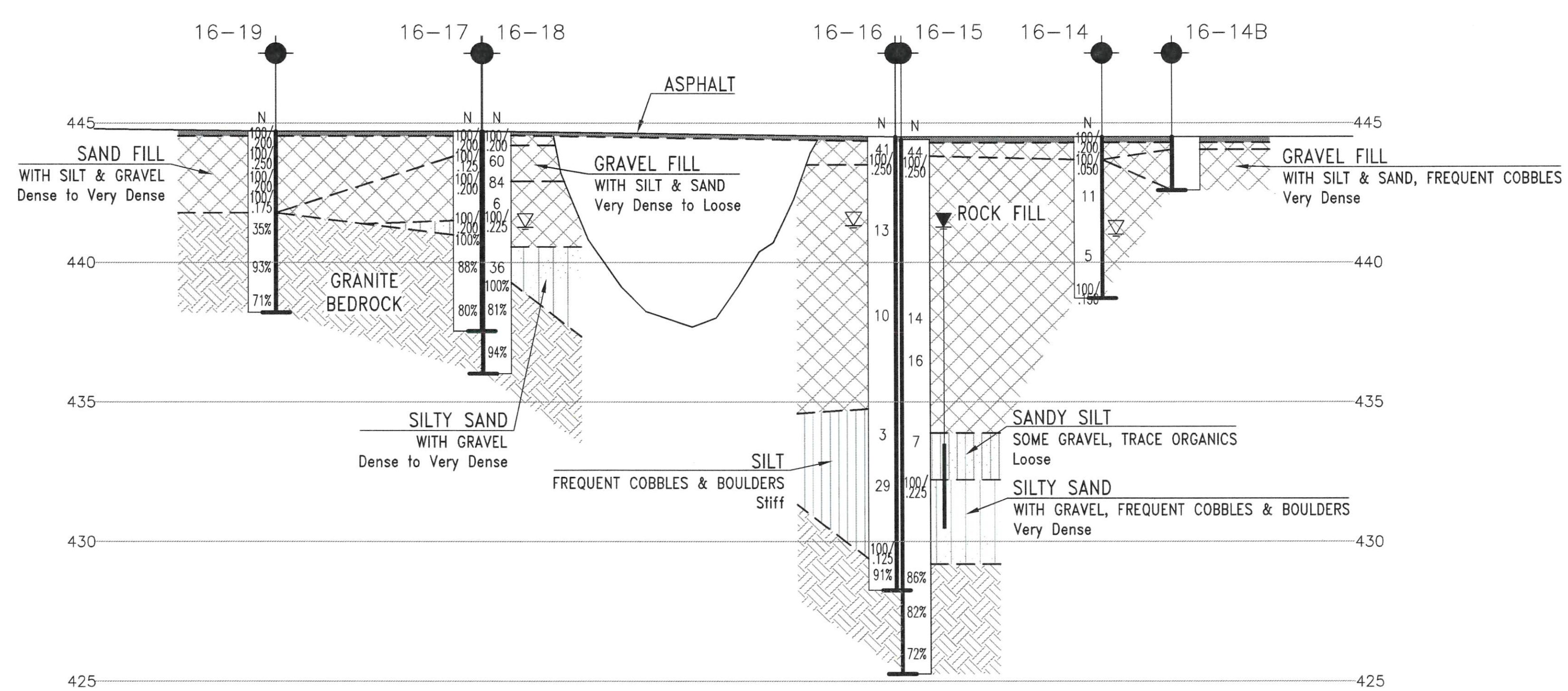

PROFILE

10 0 10 20m H 1:500
5 0 5 10m V 1:250

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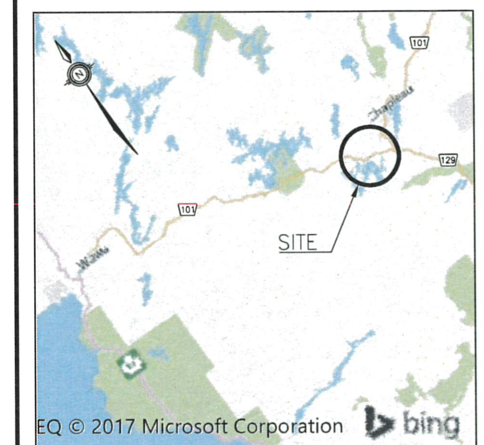


KEYPLAN

LEGEND

	Borehole
	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
16-14	444.5	5 293 164.3	343 042.0
16-14B	444.5	5 293 161.4	343 046.1
16-15	444.4	5 293 174.6	343 031.7
16-16	444.5	5 293 168.2	343 026.6
16-17	444.7	5 293 191.1	343 006.7
16-18	444.7	5 293 186.5	343 003.5
16-19	444.7	5 293 196.0	342 992.2
17-01	443.6	5 293 151.5	343 034.1
17-04	443.8	5 293 183.1	342 992.2

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
3) Borehole locations are shown in MTM Zone 13 coordinates.

GEOCRES No. 41O-26

REVISIONS	DATE	BY	DESCRIPTION

DESIGN JG	CHK -	CODE	LOAD	DATE OCT 2018	
DRAWN MFA	CHK JG	SITE 46-015	STRUCT	DWG 1	

FILENAME: H:\Drafting\13000\13624\TED-13624-BHPP-SL.dwg
PLOTDATE: 10/25/2018 12:01 PM

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

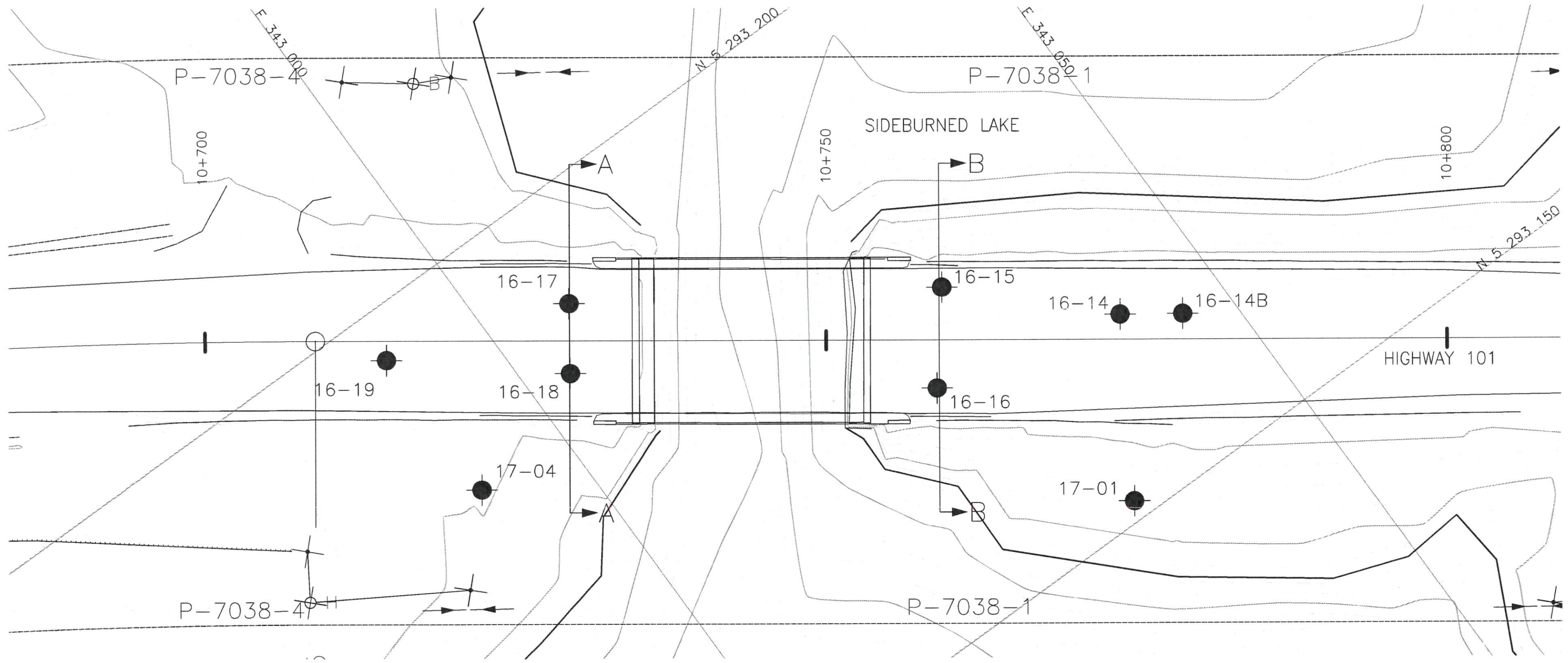
CONT No
GWP No 5144-10-00

HIGHWAY 101
SIDEBURNED LAKE
BRIDGE REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

McINTOSH PERRY

THURBER ENGINEERING LTD.



PLAN

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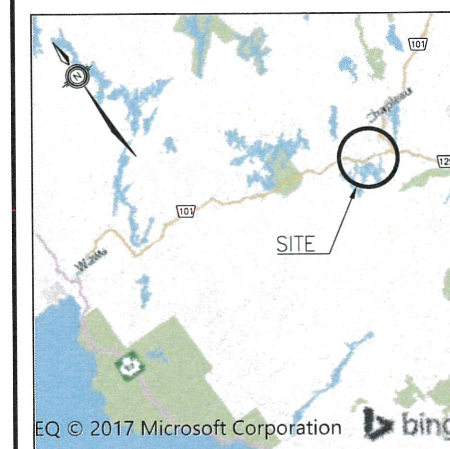
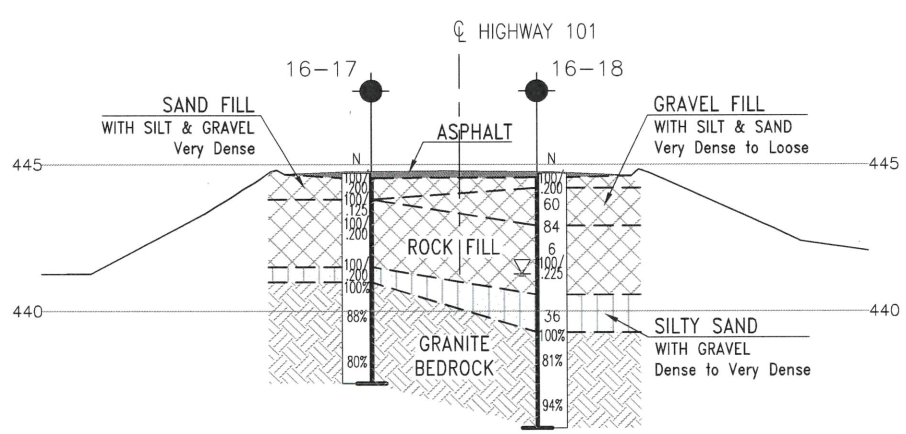


KEYPLAN



SECTION A-A

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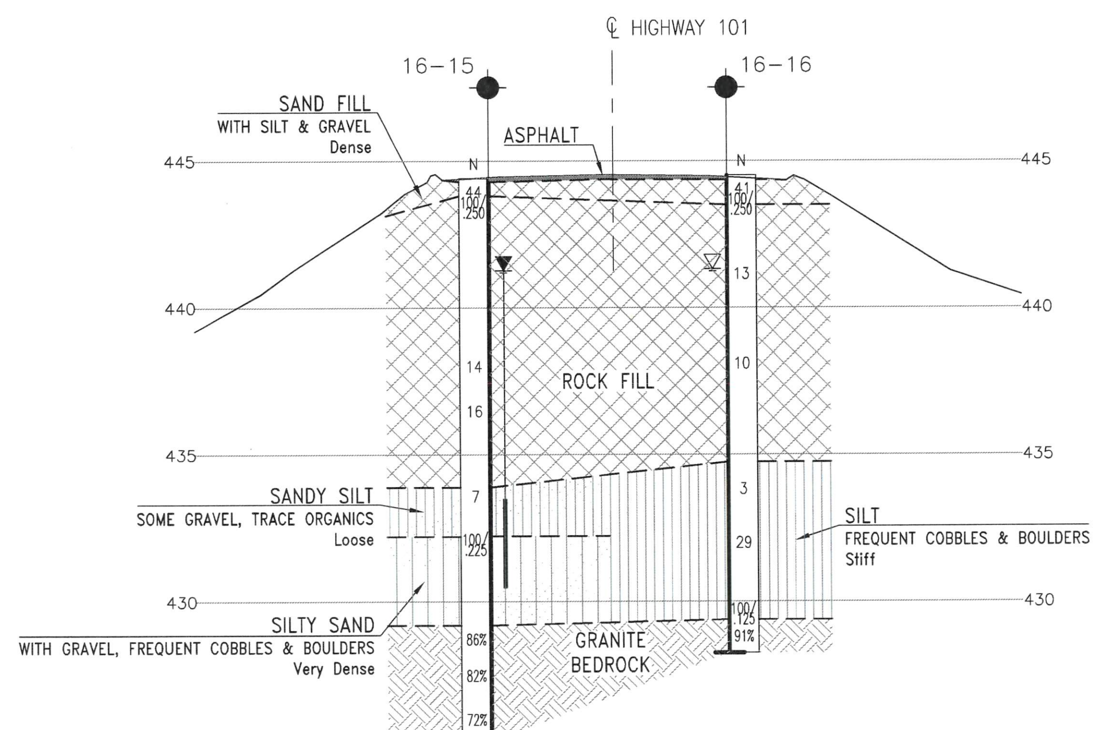


SECTION B-B

5 0 5 10m
SCALE 1:250

L E G E N D

Symbol	Description
●	Borehole
⊕	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
∇	Water Level
▼	Head Artesian Water
┃	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
16-14	444.5	5 293 164.3	343 042.0
16-14B	444.5	5 293 161.4	343 046.1
16-15	444.4	5 293 174.6	343 031.7
16-16	444.5	5 293 168.2	343 026.6
16-17	444.7	5 293 191.1	343 006.7
16-18	444.7	5 293 186.5	343 003.5
16-19	444.7	5 293 196.0	342 992.2
17-01	443.6	5 293 151.5	343 034.1
17-04	443.8	5 293 183.1	342 992.2

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
3) Borehole locations are shown in MTM Zone 13 coordinates.

GEOCRES No. 41O-26

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN JG	CHK –	CODE	LOAD	DATE OCT 2018	
DRAWN MFA	CHK JG	SITE 46-015	STRUCT	DWG 2	

FILENAME: H:\Drafting\13000\13624\TED-13624-BHPP-SL.dwg
PLOTDATE: 10/25/2018 12:01 PM

METRIC	DISTRICT
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN	CONT. No. 2018–XXXX WP No. 5837–05–01

HIGHWAY 101
SIDEBURNED RIVER BRIDGE
BRIDGE REPLACEMENT

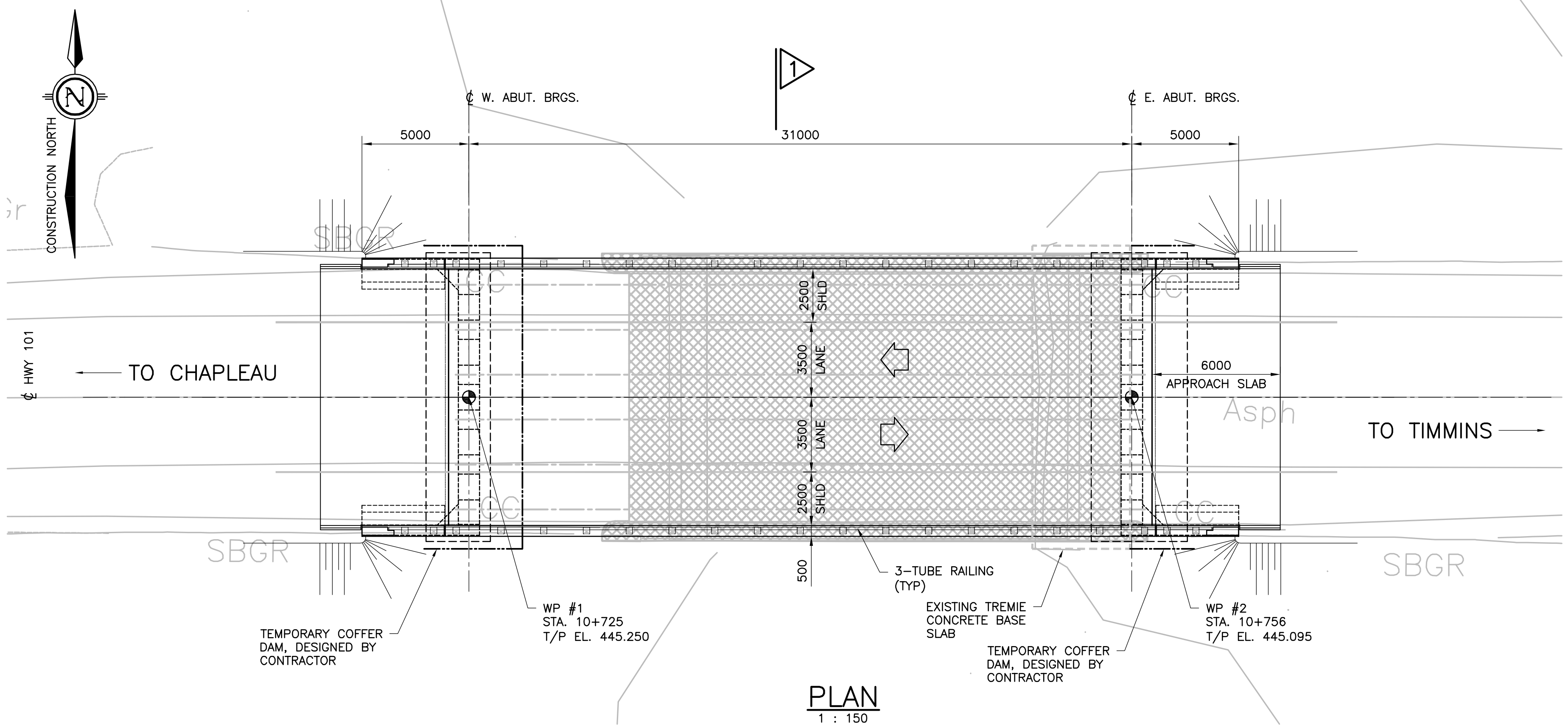
GENERAL ARRANGEMENT

SHEET 59

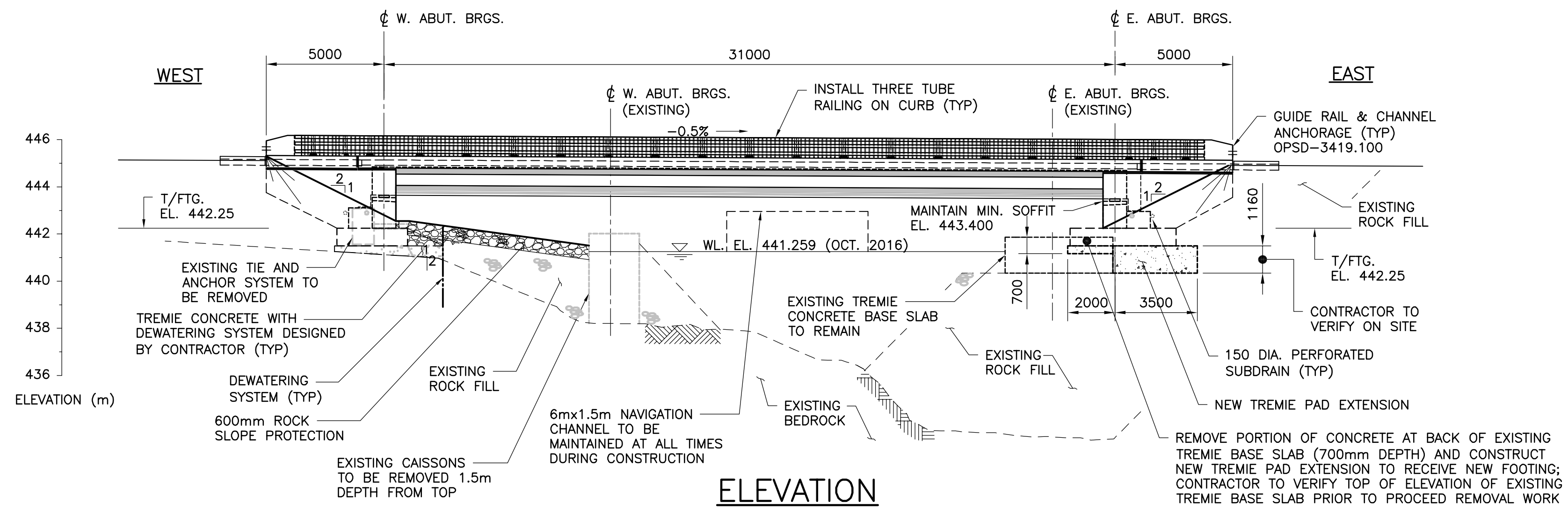
McINTOSH PERRY

LEGEND:

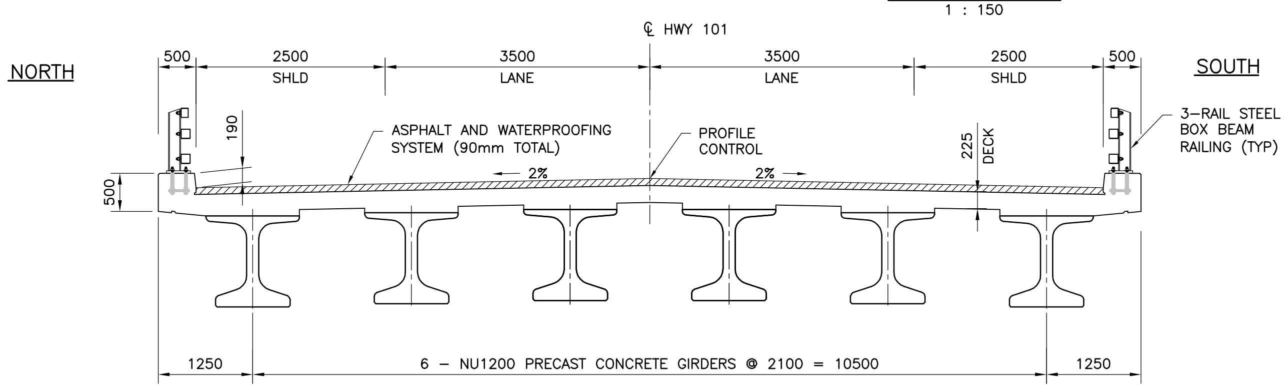
- CONCRETE AND ASPHALT REMOVAL
- NEW ASPHALT



PLAN
1 : 150



ELEVATION
1 : 150



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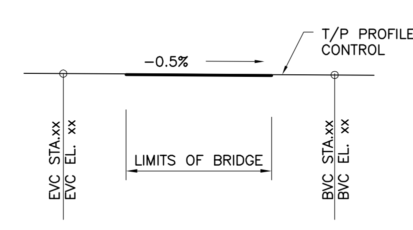


PROFILE OF HWY 101
N.T.S.

BENCHMARK:

300 BM 443.172 SPK/W E ROOT 200 SPRUCE
301 BM 445.216 RPL IN ROCK OUTCROP
302 BM 451.000 CC ON CO STEP

GENERAL NOTES:

1. CLASS OF CONCRETE: 30 MPa
2. CLASS OF CONCRETE FOR PRECAST GIRDERS ARE GIVEN ON PRESTRESSED GIRDER DRAWINGS.

CLEAR COVER TO REINFORCING STEEL:
- –FOOTING 100 ± 25
- –DECK TOP 70 ± 20
- BOTTOM 40 ± 10
- –PIER CAP 70 ± 10
- –REMAINDER 70 ± 20

UNLESS OTHERWISE NOTED

3. REINFORCING STEEL:
- REINFORCING STEEL SHALL BE GRADE 400W.
- UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL BARS SHALL BE CLASS B.
- STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa, UNLESS OTHERWISE SPECIFIED,
- BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
- BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12–1, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES:

1. CONTRACTOR SHALL OBTAIN LOCATES PRIOR TO PROCEEDING WITH CONSTRUCTION OPERATIONS.
2. THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESS FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESS ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL MAKE ADJUSTMENTS TO SUIT.
3. THE ROADWAY WILL BE CLOSED FOR THE FULL DURATION OF THE BRIDGE CONSTRUCTION.
4. INFORMATION OF EXISTING STRUCTURE SHOWN WAS TAKEN FROM THE ORIGINAL DESIGN DRAWINGS. THE CONTRACTOR SHALL VERIFY ALL RELEVANT DIMENSIONS, ELEVATIONS, STATIONS AND DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE DESIGN ENGINEER PRIOR TO PROCEEDING WITH THE CONSTRUCTION OF THE NEW BRIDGE.
5. BACKFILL SHALL NOT BE PLACED BEHIND THE ABUTMENTS UNTIL THE DECK SLAB IS IN PLACE AND HAS REACHED 70% OF ITS DESIGN STRENGTH. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.
6. THE CONTRACTOR IS RESPONSIBLE FOR DEVELOPING AND PROVIDING TEMPORARY SUPPORT SYSTEMS AND DEWATERING SYSTEMS FOR THE SAFE REMOVAL OF THE EXISTING STRUCTURE AND THE CONSTRUCTION OF THE NEW ABUTMENTS AND THE NEW SUPERSTRUCTURE AS SHOWN ON THE DRAWINGS.
7. THE CONTRACTOR SHALL PROVIDE DEBRIS PLATFORM AND NECESSARY CONTAINMENT MEASURES TO COLLECT FALLING CONCRETE AND CONSTRUCTION DEBRIS SUCH THAT NO DEBRIS OR MATERIALS RESULTING FROM THE STRUCTURE REMOVAL AND RECONSTRUCTION WORK FALLS ON THE WATERWAY BELOW OR OTHER AREAS ON THE BRIDGE SITE.

LIST OF ABBREVIATIONS:

T/CONC.	DENOTES TOP OF CONCRETE
EL.	DENOTES ELEVATION
BRGS	DENOTES BEARINGS
ABUT.	DENOTES ABUTMENT
TYP	DENOTES TYPICAL
WP	DENOTES WORKING POINT
STA.	DENOTES STATION
NTS	DENOTES NOT TO SCALE
T/FTG	DENOTES TOP OF FOOTING

LIST OF DRAWINGS:

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS AND SOIL STRATA
3. SOIL STRATA
4. CONSTRUCTION STAGING (MODULAR BRIDGE
5. EXISTING STRUCTURE REMOVAL
6. FOUNDATION LAYOUT
7. FOOTING REINFORCEMENT
8. ABUTMENT & WINGWALL DIMENSIONS
9. ABUTMENT & WINGWALL REINFORCEMENT
10. BEARING DETAILS
11. PRESTRESSED NU GIRDER I (SSD)
12. PRESTRESSED NU GIRDER II (SSD)
13. DECK DIMENSIONS
14. DECK REINFORCEMENT
15. APPROACH SLAB
16. THREE TUBE RAILING ON CURB
17. CONCRETE END WALL FOR BOX BEAM RAILING
18. STANDARD DETAILS

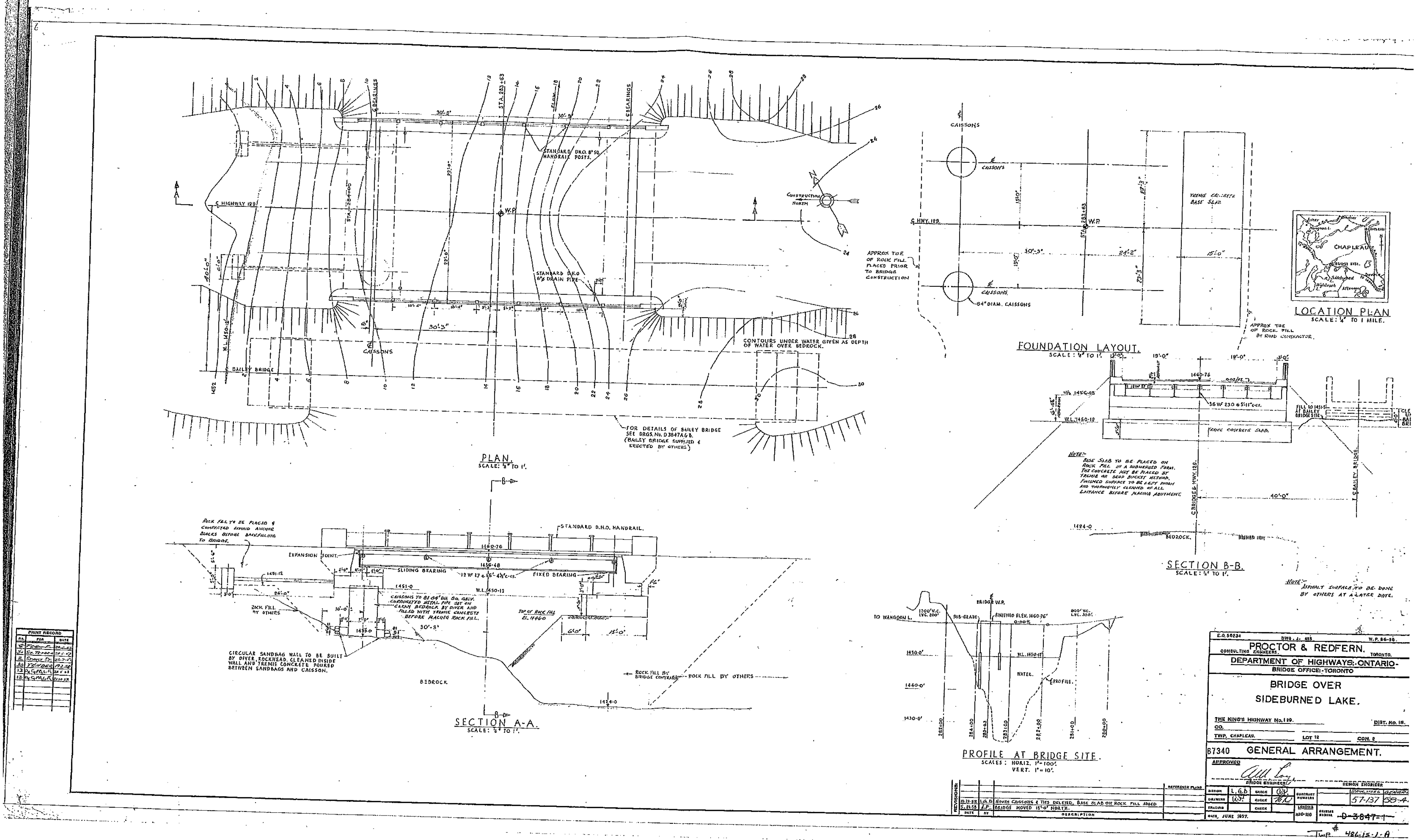
REVISIONS

DESCRIPTION

DESIGN AS	CHK TT	CODE CHBDC–14	LOAD CL–625–ONT		DATE SEP/17
DRAWN HCG	CHK AS	SITE 46–015	STRUCT	SCHEME	DWG 01

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

CAD FILE LOCATION AND NAME: \\MCINTOSHDC\mpdocuments\01 Project – Proposals\2016 Jobs\KM\0KM–16–7040 – MTO NER – 3 Str Replace + 1 Str Rehab Hwy 101 & 129\12 CAD\8 Contract Drawing\Structural\BR–03_46–015 Sideburned\S16–7040_BR–03_001GA.dwg
MODIFIED: 9/28/2017 4:22:54 PM BY: G.PENNY
DATE PLOTTED: 10/4/2017 10:12:00 AM BY: GREG PENNY
MINISTRY OF TRANSPORTATION, ONTARIO
PR–D–707 88–05


PLAN.
SCALE: 1/8" TO 1'.
C HIGHWAY 129
STA. 283+63
STA. 284+00
BEARINGS
30'-3"
22'-6"
20'-0"
W.P.
STANDARD D.H.O. 8" SQ. HANDRAIL POSTS.
STANDARD D.H.O. 6" DRAIN PIPE
CAISSONS
BAILEY BRIDGE
W.L. 1450-12
CONSTRUCTION NORTH
CONTOURS UNDER WATER GIVEN AS DEPTH OF WATER OVER BEDROCK.
FOR DETAILS OF BAILEY BRIDGE SEE DRGS. No. D3847A & B.
(BAILEY BRIDGE SUPPLIED & ERECTED BY OTHERS)
FOUNDATION LAYOUT.
SCALE: 1/8" TO 1'.
CAISSONS
TREMIE CONCRETE BASE SLAB
C HWY. 129
84" DIAM. CAISSONS
APPROX TOE OF ROCK FILL PLACED PRIOR TO BRIDGE CONSTRUCTION
APPROX TOE OF ROCK FILL BY ROAD CONTRACTOR.
LOCATION PLAN
SCALE: 1/4" TO 1 MILE.
CHAPLEAU
SECTION A-A.
SCALE: 1/8" TO 1'.
STANDARD D.H.O. HANDRAIL.
ROCK FILL TO BE PLACED & COMPACTED AROUND ANCHOR BLOCKS BEFORE BACKFILLING TO BRIDGE.
EXPANSION JOINT.
SLIDING BEARING
FIXED BEARING
1460-76
1456-48
W.L. 1450-12
1451-0
12 W 27 @ 5'-4 1/2" c-cs.
CAISSONS TO BE 84" DIA. 8G. GALV. CORRUGATED METAL PIPE SET ON CLEAN BEDROCK BY DIVER AND FILLED WITH TREMIE CONCRETE BEFORE PLACING ROCK FILL.
TOP OF ROCK FILL EL. 1446-0
ROCK FILL BY OTHERS
CIRCULAR SANDBAG WALL TO BE BUILT BY DIVER. ROCKHEAD CLEANED INSIDE WALL AND TREMIE CONCRETE POURED BETWEEN SANDBAGS AND CAISSON.
BEDROCK
ROCK FILL BY BRIDGE CONTRACTOR
ROCK FILL BY OTHERS
1424-0
SECTION B-B.
SCALE: 1/4" TO 1'.
36 W 230 @ 5'-11" c-cs.
TREMIE CONCRETE SLAB.
C BRIDGE & HWY. 129.
C BAILEY BRIDGE
40'-0"
BEDROCK.
NOTE:- BASE SLAB TO BE PLACED ON ROCK FILL IN A SUBMERGED FORM. THE CONCRETE MAY BE PLACED BY TREMIE OR DROP BUCKET METHOD. FINISHED SURFACE TO BE LEFT ROUGH AND THOROUGHLY CLEANED OF ALL LAITANCE BEFORE PLACING ABUTMENT.
NOTE:- ASPHALT SURFACE TO BE DONE BY OTHERS AT A LATER DATE.
PROFILE AT BRIDGE SITE.
SCALES: HORIZ. 1" = 100'. VERT. 1" = 10'.
TO WANGOON L.
BRIDGE W.P.
FINISHED ELEV. 1460-76'
WATER.
PROFILE.
REVISIONS
L.G.B. SOUTH CAISSONS & TIES DELETED, BASE SLAB ON ROCK FILL ADDED
J.P. BRIDGE MOVED 15'-0" NORTH.
PRINT RECORD
E.O. 56234
DWG. J- 485
W.P. 86-56.
PROCTOR & REDFERN.
CONSULTING ENGINEERS.
TORONTO.
DEPARTMENT OF HIGHWAYS:-ONTARIO-
BRIDGE OFFICE:-TORONTO
BRIDGE OVER
SIDEBURNED LAKE.
THE KING'S HIGHWAY No. 129.
DIST. No. 18.
CO.
TWP. CHAPLEAU.
LOT 12
CON. 2
67340 GENERAL ARRANGEMENT.
APPROVED
BRIDGE ENGINEER
DESIGN ENGINEER
DESIGN L.G.B. CHECK
DRAWING CHECK
TRACING CHECK
DATE, JUNE 1957.
CONTRACT NUMBERS
57-137
LOADING H20-S16
D-3847-1
TWP # 486-15-1-A

APPENDIX B

RECORD OF BOREHOLE SHEETS

THURBER

SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of fragments of decayed organic matter
Till	unstratified glacial deposit which may include particles ranging in sizes from clay to boulder
Fill	material below the surface identified as placed by humans (excluding buried services)

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

Desiccated	having visible signs of weathering by oxidization of clay materials, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

RECOVERY:

For soil samples, the recovery is recorded as the length of the soil sample recovered.

N-VALUE:

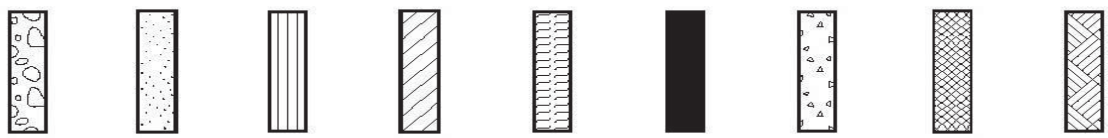
Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 63.5 kg hammer falling 0.76 m, required to drive a 50 mm O.D. split spoon sampler 0.3 m into undisturbed soil. For samples where insufficient penetration was achieved and N-value cannot be presented, the number of blows are reported over the sampler penetration in millimetres (e.g. 50/75).

DYNAMIC CONE PENETRATION TEST (DCPT):

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an “A” size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The DCPT is used as a probe to assess soil variability.

THURBER

STRATA PLOT:
Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Bedrock

TEXTURING CLASSIFICATION OF SOILS

Classification	**Particle Size**
Boulders	Greater than 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	Less than 0.002 mm

SAMPLE TYPES

SS	Split spoon samples
ST	Shelby tube or thin wall tube
DP	Direct push sample
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ etc.	Rock core sample obtained with the use of standard size diamond coring equipment

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	**Undrained Shear Strength (kPa)**
Very Soft	12 or less
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

NOTE: Clay sensitivity is defined as the ratio of the undisturbed strength over the remolded strength.

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

Descriptive Term	**SPT "N" Value**
Very Loose	Less than 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

THURBER

MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note - W_L= Liquid Limit

THURBER

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock materials.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structures are preserved.

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1 m in length or larger, as a percentage of total core length
Unconfined Compressive Strength: (UCS)	Axial stress required to break the specimen.
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 to 2 m
Medium bedded	0.2 to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 to 60 mm
Laminated	6 to 20 mm
Thinly laminated	Less than 6 mm

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)
Extremely Strong	Greater than 250
Very Strong	100 – 250
Strong	50 – 100
Medium Strong	25 – 50
Weak	5 – 25
Very Weak	1 – 5
Extremely Weak	0.25 – 1

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-14

METRIC

GWP#	5144-10-00	LOCATION	Hwy 101 - Sideburned Lake Bridge N 5 293 164.3 E 343 042.0	ORIGINATED BY	CM
HWY	101	BOREHOLE TYPE	HSA / CME 75 Truck Mount	COMPILED BY	JM
DATUM	Geodetic	DATE	2016.10.30 - 2016.10.30	CHECKED BY	SP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
444.5													
0.0	**200 mm ASPHALT**												
0.2	SAND with Silt and Gravel Brown Very Dense **FILL**		1	SS	100/ 200mm		444	○		41	52	7 (SI+CL)	
443.7 / 0.8	- Very difficult augering below 0.3 m		2	SS	100/ 50mm								
	ROCK FILL - Auger refusal at 0.8 m, switch to NW Casing - Boulder from 0.8 m to 1.2 m - Boulder from 1.5 to 1.7 m						443						
	GRAVEL infill from 1.7 m to 2.4 m, grey, compact		3	SS	11								
	- Boulder from 2.4 m to 2.6 m						442						
	- Boulder from 2.9 m to 3.2 m												
	- Boulder from 3.4 m to 3.7 m					∇	441						
	GRAVEL with Sand infill from 3.7 m to 5.2 m, brown, loose		4	SS	5		440	○					
	- Boulder from 5.2 m to 5.4 m						439						
438.7 / 5.8			5	SS	100/ 150mm								
	End of borehole at 5.8 m on probable boulder in rockfill Groundwater level was measured at 3.5 m BGS (Elev. 441.0) on 2016.11.01												

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-14B

METRIC

GWP#	5144-10-00	LOCATION	Hwy 101 - Sideburned Lake Bridge N 5 293 161.4 E 343 046.1	ORIGINATED BY	CM
HWY	101	BOREHOLE TYPE	HSA / CME 75 Truck Mount	COMPILED BY	JM
DATUM	Geodetic	DATE	2016.11.01 - 2016.11.01	CHECKED BY	SP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
444.5							
0.0	**210 mm ASPHALT**						
444.1 / 0.2	SAND with Silt and Gravel, Brown, Dense, **FILL**	1	AS		444	~0	
0.5	GRAVEL with Silt and Sand, Brown, Very Dense, **FILL** - Frequent cobbles and boulders below 0.9 m	2	AS		443	~0	62 30 8 (SI+CL)
442.6 / 1.9	End of Borehole. Auger refusal on probable boulder. Borehole backfilled with cuttings						

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; STRAT PLOT.

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-15

METRIC

GWP# 5144-10-00	LOCATION Hwy 101 - Sideburned Lake Bridge N 5 293 174.6 E 343 031.7		ORIGINATED BY CM
HWY 101	BOREHOLE TYPE HSA / HW Casing / NW Casing / NQ Coring		COMPILED BY JM
DATUM Geodetic	DATE 2016.10.30 - 2016.11.01		CHECKED BY SP

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT
444.4						
0.0	**130mm ASPHALT**					
0.1	SAND with Silt and Gravel, Brown, Dense, **FILL**	1	SS	44	444	○ (~0)
443.8 / 0.6	**ROCK FILL**	2	SS	100/ 250mm		
	- Auger Refusal at 0.6 m, switch to HW Casing					
	- Boulder from 1.0 m to 1.4 m				443	
	- Boulder from 1.5 m to 1.8 m					
	- Boulder from 2.1 m to 2.4 m				442	
	- Boulder from 2.6 m to 2.8 m					
	- Cobbles from 2.9 m to 3.2 m					
	- Boulder from 3.2 m to 3.4 m				441	
	- Boulder from 3.5 m to 3.7 m					
	- Frequent cobbles from 3.7 m to 4.4 m					
					440	
	- Boulder from 4.6 m to 5.1 m					
	- Frequent cobbles from 5.2 m to 6.0 m				439	
		3	SS	14	438	
					437	
	GRAVEL with Sand infill from 6.0 m to 8.5 m, dark brown, compact	4	SS	16		
					436	
	- Frequent cobbles from 8.5 m to 8.8 m					
	- Switch to NW casing					
	- Boulder from 9.0 m to 9.3 m				435	

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-15

METRIC

GWP# 5144-10-00
LOCATION Hwy 101 - Sideburned Lake Bridge N 5 293 174.6 E 343 031.7
ORIGINATED BY CM

HWY 101
BOREHOLE TYPE HSA / HW Casing / NW Casing / NQ Coring
COMPILED BY JM

DATUM Geodetic
DATE 2016.10.30 - 2016.11.01
CHECKED BY SP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page						
	- Boulder from 10.1 m to 10.3 m						
433.9 / 10.5	**Sandy SILT (ML)** some Gravel trace Organics Grey Loose	5	SS	7	434		10 24 58 8
	- Boulder from 11.3 m to 11.5 m				433		
432.2 / 12.2	**SILTY SAND (SM)** with Gravel, frequent Cobbles and Boulders Grey Very Dense	6	SS	100/ 225mm	432		Chemical Testing
	- Boulder from 12.7 m to 13.0 m						
	- Frequent cobbles from 13.0 m to 13.4 m				431		
	- Boulder from 13.6 m to 13.9 m						
	- Boulder from 13.9 m to to 14.1 m						
	- Boulder from 14.2 m to 14.4 m				430		
429.2 / 15.2	Casing refusal				429		
	Bedrock Granite Fresh Moderately Bedded Grey	1	RUN				RUN #1 TCR=100% SCR=100% RQD=86%
		2	RUN		428 / 427		RUN #2 TCR=100% SCR=95% RQD=82%
		3	RUN		426		RUN #3 TCR=100% SCR=100% RQD=72%
425.3 / 19.1	End of Borehole Groundwater level was measured in piezometer at 3.1 m BGS (Elev. 441.3) on 2016.11.04						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m3

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-16

METRIC

GWP# 5144-10-00	LOCATION Hwy 101 - Sideburned Lake Bridge N 5 293 168.2 E 343 026.6	ORIGINATED BY CM
HWY 101	BOREHOLE TYPE HSA / HW Coring / CME 75 Truck Mount	COMPILED BY JM
DATUM Geodetic	DATE 2016.11.01 - 2016.11.01	CHECKED BY SP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
444.5									
0.0	**150mm ASPHALT**								
0.2	SAND with Silt and Gravel Brown Dense **FILL** Difficult augering below 0.6 m, switch to HW Casing at 1.0 m	1	SS	41		444	o		
		2	SS	100/ 250mm			o		
443.4 1.0	**ROCK FILL** - Boulder from 1.0 m to 1.6 m - Boulder from 1.8 m to 2.3 m - Boulder from 2.6 m to 3.0 m - Frequent cobbles from 3.0 m to 3.8 m GRAVEL infill from 3.0 m to 3.8 m, grey, compact	3	SS	13	▽	443 442 441			
	- Boulder from 3.8 m to 4.0 m - Boulder from 4.1 m to 4.3 m - Boulder from 4.5 m to 4.8 m - Boulder from 5.1 m to 5.3 m - Frequent cobbles from 5.5 m to 6.1 m Gravel infill from 6.1 m to 6.9 m, grey, compact	4	SS	10		440 439 438			
	- Boulder from 6.9 m to 7.1 m - Boulder from 7.2 m to 7.5 m - Boulder from 7.6 to 8.0 m - Boulder from 8.2 m to 9.1 m - Switch to NW Casing - Boulder from 9.5 m to 9.8 m					437 436 435			
434.7 9.8	**SILT (ML)**								

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-16

METRIC

GWP# 5144-10-00 | LOCATION Hwy 101 - Sideburned Lake Bridge N 5 293 168.2 E 343 026.6 | ORIGINATED BY CM

HWY 101 | BOREHOLE TYPE HSA / HW Coring / CME 75 Truck Mount | COMPILED BY JM

DATUM Geodetic | DATE 2016.11.01 - 2016.11.01 | CHECKED BY SP

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (kPa) / DCPT	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT (ML) Grey Stiff				434			
		5	SS	3			o H	0 1 82 17
	- Clayey				433	+ 4.0; + 2.5 (field vane)		
		6	SS	29	432		o	0 5 76 19
	- Boulder from 13.1 m to 13.3 m							
	- Boulder from 13.6 m to 14.2 m				431			
	- Frequent cobbles from 14.3 m to 14.9 m				430			
429.3		7	SS	100/ 125mm				
15.1	**Bedrock** Granite Fresh Moderately Bedded Grey	1	RUN		429			RUN #1 TCR=100% SCR=0.96% RQD=91%
428.2 16.2	End of Borehole Groundwater level was measured at 3.2 m BGS (Elev. 441.3) on 2016.11.02							

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; UNIT WEIGHT γ kN/m^3

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 16-17

METRIC

GWP# 5144-10-00 | LOCATION Hwy 101 - Sideburned Lake Bridge N 5 293 191.1 E 343 006.7 | ORIGINATED BY CM

HWY 101 | BOREHOLE TYPE HSA / NW Casing / NQ Coring | COMPILED BY JM

DATUM Geodetic | DATE 2016.11.04 - 2016.11.04 | CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
444.7							
0.0	**165mm ASPHALT**						
0.2	SAND with Silt and Gravel, Very Dense, Brown, **FILL**	1	SS	100/ 200mm			30 58 12 (SI+CL)
443.8 / 0.9	**ROCK FILL** - Auger refusal at 0.9 m, switch to NW Casing	2	SS	100/ 125mm		444	
		3	SS	100/ 200mm		443	
	- Boulder from 2.3 m to 2.5 m						
	- Boulder from 2.6 m to 2.9 m					442	
	- Boulder from 3 m to 3.2 m						
441.5 / 3.2	**Silty SAND (SM)** with Gravel, Brown, Very Dense	4	SS	100/ 200mm			
441.0	- Wood fragments						
3.7	**Bedrock** Granite, Fresh, Moderately Bedded, Grey	1	RUN			441	RUN #1 TCR=100% SCR=100% RQD=100%
		2	RUN			440	RUN #2 TCR=100% SCR=97% RQD=88%
		3	RUN			439 / 438	RUN #3 TCR=100% SCR=97% RQD=80%
437.6 / 7.2	End of Borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 16-18

METRIC

GWP#	5144-10-00	LOCATION	Hwy 101 - Sideburned Lake Bridge N 5 293 186.5 E 343 003.5	ORIGINATED BY	CM
HWY	101	BOREHOLE TYPE	HSA / NW Casing / NQ Coring	COMPILED BY	JM
DATUM	Geodetic	DATE	2016.11.03 - 2016.11.03	CHECKED BY	SP

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
444.7								
0.0	**150mm ASPHALT**							
0.2 / 444.2 / 0.5	SAND with Silt and Gravel Grey Very Dense **FILL**		1	SS	100/ 200mm		444	23 66 11 (SI+CL)
	GRAVEL with Silt and Sand Very Dense to Loose Grey **FILL** - Very difficult augering from 0.5 m to 0.8 m - Switch to NW Casing at 1.5 m		2	SS	60			
442.9 / 1.8	**ROCKFILL** - Boulder from 1.8 m to 2.1 m GRAVEL infill 2.1 m to 3.4 m, grey, loose		3	SS	84		443	
			4	SS	6		442	
	- Boulder from 3.4 m to 4.1 m		5	SS	100/ 225mm	∇	441	
440.6 / 4.1	**Silty SAND (SM)** with Gravel Dense Grey		6	SS	36		440	Chemical Testing
439.3 / 5.4	**Bedrock** Granite Fresh Moderately Weathered Grey		1	RUN			439	RUN #1 TCR=100% SCR=100% RQD=100%
			2	RUN			438	RUN #2 TCR=98% SCR=98% RQD=81%
			3	RUN			437	RUN #3 TCR=100% SCR=100% RQD=94%
436.0 / 8.7	End of borehole Groundwater level was measured at 3.44 m BGS (Elev. 441.3) on 2016.11.03							

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 16-19

METRIC

GWP# 5144-10-00
LOCATION Hwy 101 - Sideburned Lake Bridge N 5 293 196.0 E 342 992.2
ORIGINATED BY CM
HWY 101
BOREHOLE TYPE HSA / CME 75 Truck Mount
COMPILED BY JM
DATUM Geodetic
DATE 2016.11.03 - 2016.11.03
CHECKED BY SP

Column headings: SOIL PROFILE (ELEV / DEPTH; DESCRIPTION; STRAT PLOT); SAMPLES (NUMBER; TYPE; "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
444.7									
0.0	**165 mm ASPHALT**								
0.2	SAND with Silt and Gravel, Very Dense, Brown, **FILL** - Difficult Augering from 0.5 m to 0.7 m	1	SS	100/ 200mm					
					444				
		2	SS	100/ 250mm		25	63	12 (SI+CL)	
		3	SS	100/ 200mm	443				
	- Difficult Augering from 1.8 m to 2.2 m								
		4	SS	100/ 175mm					
	Auger refusal at 2.6 m				442				
441.8	- Boulder from 2.6 m to to 2.7 m								
2.9	**Bedrock** Granite, Moderately weathered, Fresh, Grey	1	RUN						
					441				
					440				
		2	RUN						
					439				
		3	RUN						
438.2									
6.5	End of borehole. Borehole dry prior to coring								

ONTMT4S 13624 - 101 AND 129 - SIDEBURNED LAKE.GPJ 2012TEMPLATE(MTO).GDT 24/10/18

+ 3, × 3 : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

APPENDIX C

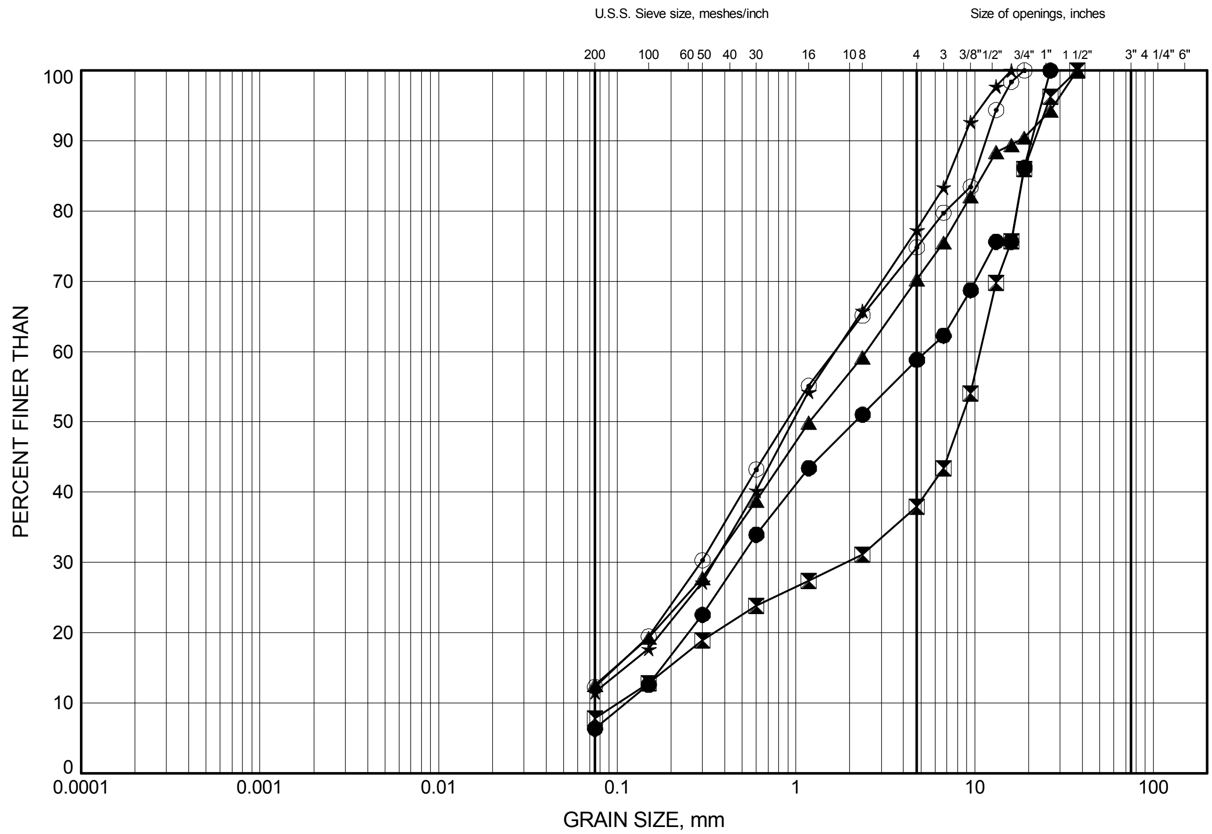
LABORATORY TEST RESULTS

Sideburned Lake Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C1

FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-14	0.40	444.11
⧗	16-14B	1.22	443.30
▲	16-17	0.46	444.29
★	16-18	0.34	444.40
⊙	16-19	0.88	443.82

Date October 2018

GWP# 5144-10-00

THURBER

Prep'd CM

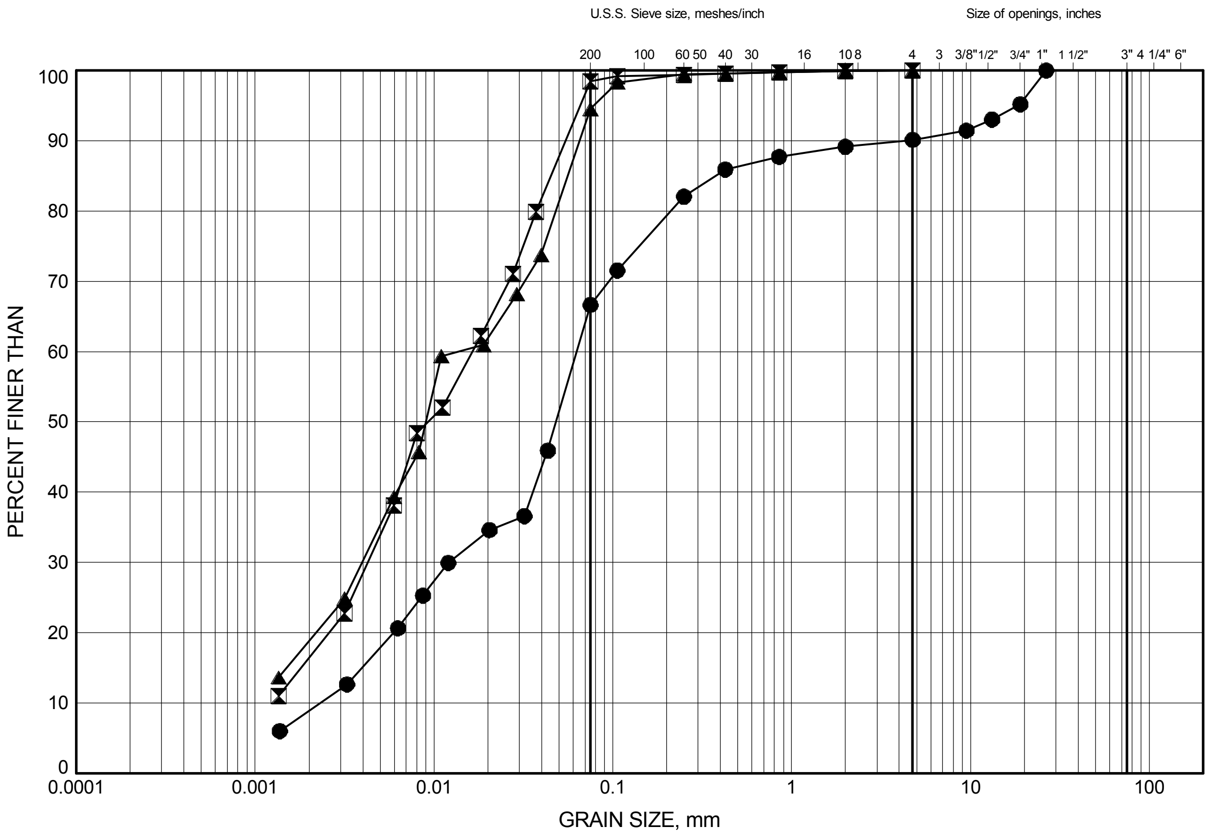
Chkd. SP

Sideburned Lake Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C2

Silt to Sandy Silt



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-15	10.82	433.62
⧗	16-16	10.67	433.78
▲	16-16	12.50	431.95

Date October 2018

GWP# 5144-10-00

THURBER

Prep'd CM

Chkd. SP

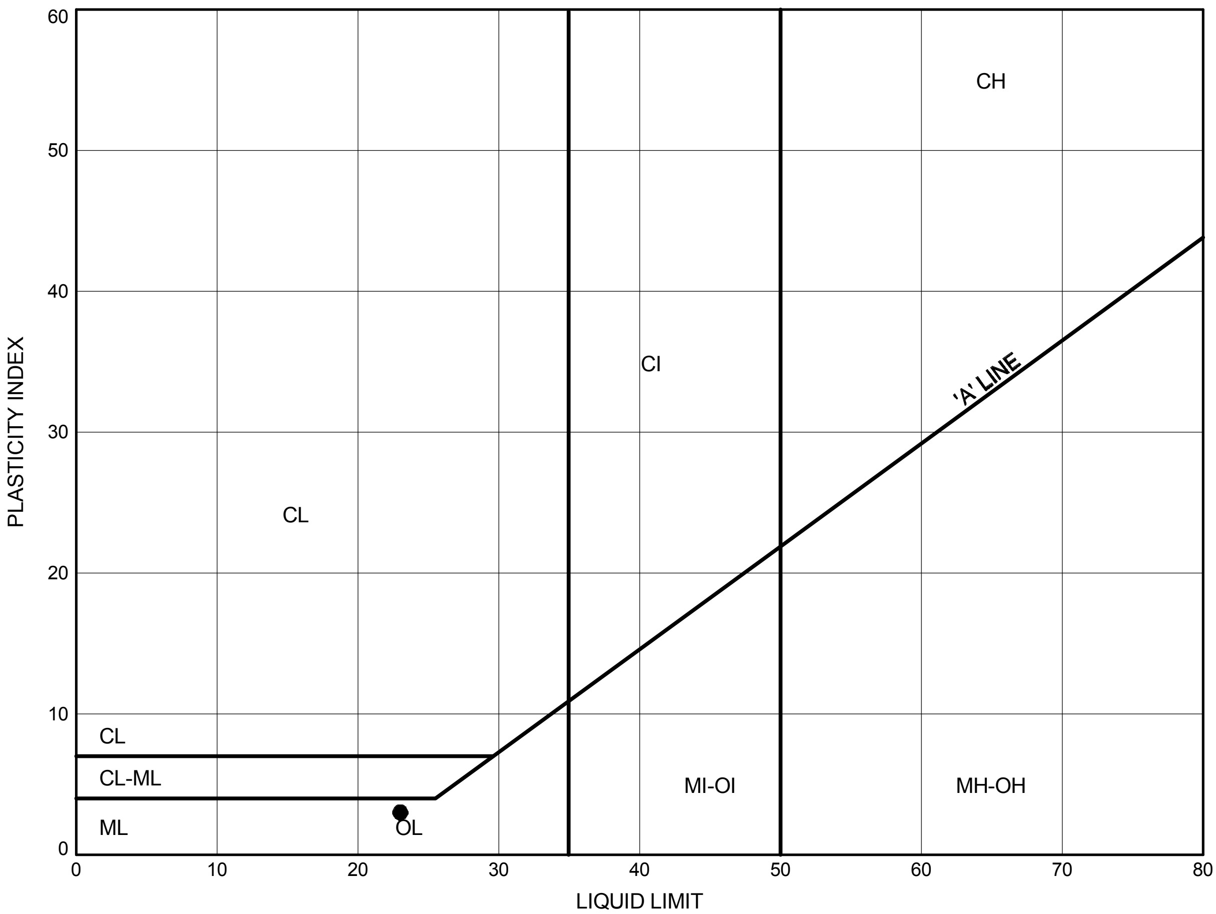
GRAIN SIZE DISTRIBUTION - THURBER 13624 - 101 AND 129 - SIDEBURNED LAKE.GPJ 24/10/18

Sideburned Lake Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE C3

Silt to Sandy Silt



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-16	10.67	433.78

Date October 2018

GWP# 5144-10-00

THURBER

Prep'd CM

Chkd. SP

PARACEL LABORATORIES LTD.

Order #: 1646369

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO:

Report Date: 17-Nov-2016
Order Date: 11-Nov-2016
Project Description: 13624

	Client ID:	16-1 SS2 (2'-4')	16-4 (1-4)	16-6 SS3 (5'-7')	16-8 SS4 (7'6-9'6)
	Sample Date:	21-Oct-16	23-Oct-16	27-Oct-16	28-Oct-16
	Sample ID:	1646369-01	1646369-02	1646369-03	1646369-04
	MDL/Units	Soil	Soil	Soil	Soil
Physical Characteristics					
% Solids	0.1 % by Wt.	81.8	85.3	96.7	92.0
General Inorganics					
Conductivity	5 uS/cm	109	109	385	728
pH	0.05 pH Units	7.41	6.41	7.89	7.89
Resistivity	0.10 Ohm.m	91.5	91.7	26.0	13.7
Anions					
Chloride	5 ug/g dry	16	15	159	346
Sulphate	5 ug/g dry	19	14	10	31

	Client ID:	16-15 SS6 (40-41-4)	16-18 SS6 (15-17)	-	-
	Sample Date:	31-Oct-16	03-Nov-16	-	-
	Sample ID:	1646369-05	1646369-06	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	89.1	84.1	-	-
General Inorganics					
Conductivity	5 uS/cm	171	351	-	-
pH	0.05 pH Units	7.78	6.84	-	-
Resistivity	0.10 Ohm.m	58.4	28.5	-	-
Anions					
Chloride	5 ug/g dry	24	171	-	-
Sulphate	5 ug/g dry	54	18	-	-

Stantec

Stantec Consulting Ltd
2781 Lancaster Rd, Suite 100 A&B
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 722-2799

January 18, 2017
File: 122410864

Attention: **Thurber Engineering Ltd., File #13624**

Reference: **ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core**

The table below summarizes four (4) Rock Core compressive strength results.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
BH16-15 RC-A	54'3"	65.1	One large diagonal crack through centre of core
BH16-16 RC-B	53'	125.7	One large diagonal crack through centre of core
BH16-17 RC-C	15'11'	206.4	No cones formed, vertical cracks throughout core
BH16-18 RC-D	22'4"	192.4	Well-formed cone on one end, vertical cracks through other

Sincerely,

Stantec Consulting Ltd

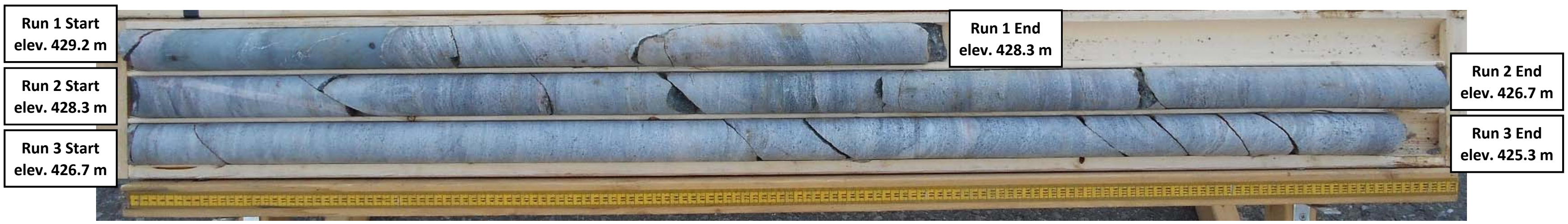
Brian Prevost

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
brian.prevost@stantec.com

Borehole 16-15

Run 1 to 3 (of 3)

Elevation 429.2 m to 425.3 m



THURBER ENGINEERING LTD.

Foundation Investigation
Highway 101 – Sideburned Lake Bridge
Site 46-015

GWP: 5144-10-00

Project No.: 13624

Borehole 16-16

Run 1 (of 1)

Elevation 429.3 m to 428.2 m

THURBER ENGINEERING LTD.

Foundation Investigation
Highway 101 – Sideburned Lake Bridge
Site 46-015

GWP: 5144-10-00

Project No.: 13624

Borehole 16-17

Run 1 to 3 (of 3)

Elevation 441.0 m to 437.6 m

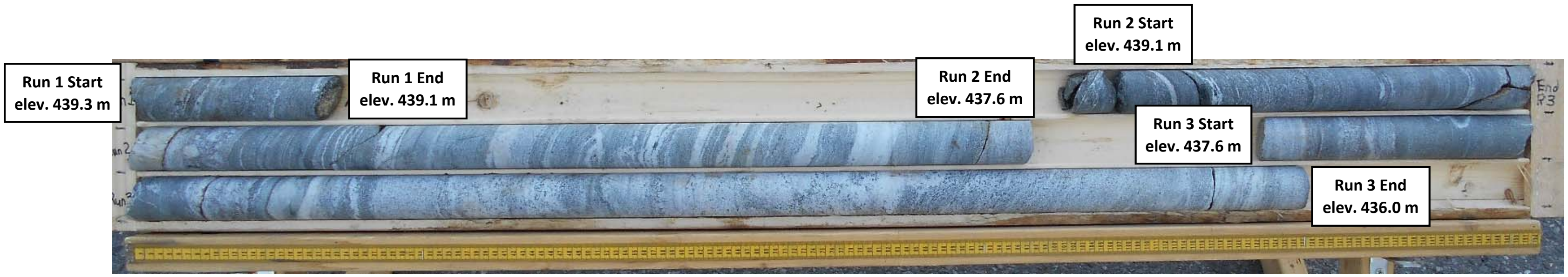
THURBER ENGINEERING LTD.

Foundation Investigation
Highway 101 – Sideburned Lake Bridge
Site 46-015

GWP: 5144-10-00

Project No.: 13624

Borehole 16-18
Run 1 to 3 (of 3)
Elevation 439.3 m to 436.0 m



THURBER ENGINEERING LTD.

Foundation Investigation
Highway 101 – Sideburned Lake Bridge
Site 46-015

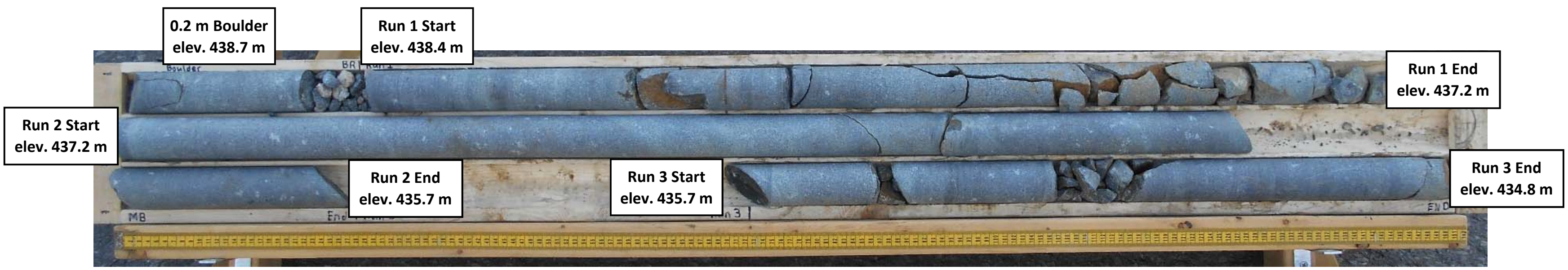
GWP: 5144-10-00

Project No.: 13624

Borehole 16-19

Run 1 to 3 (of 3)

Elevation 438.4 m to 434.8 m



THURBER ENGINEERING LTD.	Foundation Investigation Highway 101 – Sideburned Lake Bridge Site 46-015	GWP: 5144-10-00 Project No.: 13624

APPENDIX D

SELECTED PHOTOGRAPHS

Figure 1: Roadway Platform at Bridge 46-015 looking East [taken October 2016]

Figure 2: South Diversion Alignment looking West [taken October 2016]

Figure 3: Looking towards East Abutment [taken October 2016]

Figure 4: Looking west from South embankment slope [taken October 2016]

APPENDIX E

COMPARISON OF FOUNDATION ALTERNATIVES

Comparison of Bridge Type/Foundation Alternatives

Comment	Spread Footings	Caissons (Socketed into Bedrock)	Steel Piles (H-Piles, Pipe Piles)
Advantages	- Generally less costly construction then deep foundations - Accommodates abutments perched within approach fills - Requires less specialized construction installation equipment	- Higher geotechnical capacity then spread footings - Construction can continue in winter weather conditions - Reduces magnitude of excavations and limits dewatering requirements	- Higher geotechnical capacity then spread footings - Construction can continue in winter weather conditions - Likely requires less concrete than spread footings or caissons
Disadvantages	- Requires larger excavation - Requires deeper excavation to construct footing below the frost penetration depth - Will require dewatering an excavation within rockfill below the river level - Lower geotechnical resistance then deep foundations - Ineffective for resistance to uplift or overturning - Requires local availability of concrete if cast-in-place footings are used	- Higher unit cost than spread footings - Requires local availability of concrete - Specialized installation measures such as equipment, liners and drilling mud will be required - Potential difficulty in cleaning and inspecting base - Difficulty in drilling through rock fill and into bedrock	- Higher unit cost than spread footings - Requires pre-drilling through rockfill, overburden and socketed into bedrock and has potential to encounter obstructions in the native soils
Risks / Consequences	Difficulty in dewatering excavation	Difficulty in advancing through obstructions	Difficulty advancing through obstructions
Relative Cost	Moderate to High (including dewatering)	High	Moderate to High
	Generally Feasible	**Feasible**	**Recommended (Pipe Piles)**

APPENDIX F

GSC SEISMIC HAZARD CALCULATION
LIST OF REFERENCED SPECIFICATIONS
NSSP FOR ROCK DOWELS

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

December 19, 2016

Site: 47.7764 N, 83.4896 W User File Reference: Sideburned

Requested by: Chris Murray, Thurber Engineering

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.051	0.072	**0.070**	0.060	**0.049**	**0.031**	**0.016**	**0.0037**	**0.0016**	**0.040**	**0.038**

Notes. Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. ***These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.0048	0.017	0.027
Sa(0.1)	0.0081	0.026	0.041
Sa(0.2)	0.0098	0.028	0.043
Sa(0.3)	0.0089	0.026	0.038
Sa(0.5)	0.0068	0.022	0.032
Sa(1.0)	0.0034	0.013	0.020
Sa(2.0)	0.0013	0.0055	0.0095
Sa(5.0)	0.0004	0.0012	0.0021
Sa(10.0)	0.0003	0.0007	0.0010
PGA	0.0045	0.015	0.023
PGV	0.0038	0.014	0.023

References

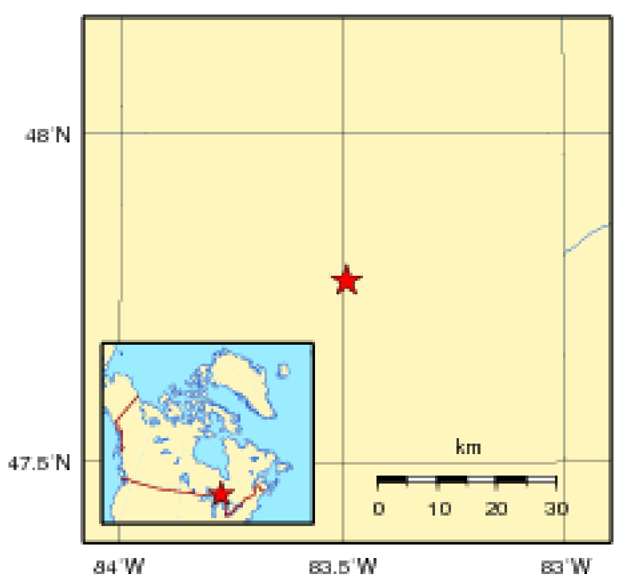
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français



LIST OF REFERENCED SPECIFICATIONS

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirements
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS.PROV 904	Construction Specification for Concrete Structures
OPSS 942	Construction Specification for Prestressed Soil and Rock Anchors
OPSS.PROV 1010	Material Specification for Aggregates-Base, Subbase, Select Subgrade, and Backfill Material
OPSS 1860	Material Specification for Geotextiles
SP 109S12	
SP 517F01	

<u>DOWELS INTO ROCK </u>– Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR THE SUPPLY, INSTALLATION AND TESTING OF DOWELS INTO ROCK FOR PIER FOOTINGS

1.0 SCOPE

The work for the above noted tender item shall be in accordance with OPSS 904, including all Special Provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the pier footing.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications, or publications:

ASTM International

D1143M Standard Test Methods for Deep Foundations Under Static Axial Compressive Load

3.0 DEFINITIONS

For the purpose of this Special Provision, the following definitions apply:

Dowels into Rock: reinforcing steel bar and non-shrink grout.

Design Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.

Quality Verification Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Working Drawings

Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.

The Contractor shall submit Working Drawings to the Contract Administrator as follows:

a) All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.

b) All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.

Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.

Working Drawings consisting of testing an installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.

Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:

a) Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.

b) Test results verifying the 28 day strength of non-shrink grout.

c) The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.

d) The procedures to verify hole length. Records of measurements that verify the hole length.

e) Records of all drilling procedures, rock conditions encountered, and installation times.

f) Test procedures for Dowels into Rock.

g) Drawings and design calculations for a suitable reaction system for the applied test loads.

h) Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.

i) Drawings and details for reference system arrangement.

j) Current calibration curves shall be provided for all gauges.

k) Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.

l) Remedial measures for unacceptable stressing results.

5.0 MATERIALS

5.01 Non-Shrink Grout

The non-shrink grout shall be an approved product from the MTO's Pre-Qualified Products List.

5.02 Anti-Washout Agent

The anti-washout agent shall be used with the non-shrink grout for the Dowels into Rock. The anti-washout agent shall be one of the following proprietary products:

1) Sikament 100 SC Anti-Washout Admixture
Sika Canada Inc.
6915 Davand Drive
Mississauga, ON, L5T 1L5
Toll Free Phone: 800-933-7452

2) Rheomac UW 450 Anti-Washout Admixture
BASF Construction Chemicals Canada Ltd (Master Builders)
1800 Clark Blvd
Brampton, ON, L6T 4M7
Toll Free Phone: 416-520-1392

5.03 Manufacturer Information

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

a) Data sheets for the non-shrink grout and anti-washout agent,

b) Technical information that proves that the non-shrink grout and anti-washout agent are compatible, and

c) installation procedures

6.0 EQUIPMENT

All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

The equipment shall not cause damage to the reinforcing steel bars.

7.0 CONSTRUCTION

7.01 Instructions to Contractor

These instructions are to be read in conjunction with the Contract Drawings.

A total of 2 test Dowels into Rock are required for the Dowels into Rock at the pier.

Dowels into rock at the pier shall be installed into sound bedrock to the specified embedment depth.

7.02 Responsibilities of the Contractor

The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.

The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.

The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 4.0.

7.03 Subsurface Conditions

Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

7.04 Construction of Holes

The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.

At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

7.05 Installation of Reinforcing Steel Bar

Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

Dowels into Rock at the pier shall be installed into sound bedrock.

Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

7.06 Grout and Anti-Washout Agent

The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

Anti-washout agent shall be used in accordance with the specifications of the manufacturer.

Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

8.0 QUALITY ASSURANCE

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.01 Qualifications

8.01.01 Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock

All work shall be performed under the direction of personnel experienced with all aspects associated with the installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.

8.01.02 Qualifications of the Quality Verification Engineer

A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.

8.01.03 Qualifications of the Design Engineer

A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

8.02 Testing Requirements

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.02.01 General Testing Requirements

Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.

The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.

The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the pier. The Dowels into Rock for testing shall be __M dowels grouted into ___ mm diameter holes filled with an approved non-shrink grout with a minimum _____ mm embedment into sound bedrock.

The Contractor shall submit Working Drawings that include proposed procedures for testing of the Dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.

The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.

The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.

The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

8.02.02 Testing Location

The Contractor shall remove all loose rock down to sound bedrock at the test location.

The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator.

If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

8.02.03 Testing Equipment

The dowels into rock will be carried out generally in accordance with the prevailing requirements of ASTM International D1143M superseded where applicable by the procedures specified in this document.

The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.

The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:

The beams shall be independently supported with the support firmly embedded in the ground.

The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.

Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

8.02.04 Testing for Dowels Into Rock, and Report

At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Jacks used for reinforcing steel bars shall have a minimum ram dimension of 152.6 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

8.02.05 Testing Loading

The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the test load of ____ kN. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

8.03 Acceptance Criteria

The following acceptance criteria apply:

a) The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at the pier footing.

b) Tests for Dowels into Rock shall have a capacity of at least ____ kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

9.0 MEASUREMENT FOR PAYMENT

For measurement purposes, a count shall be made of the number of dowels installed.

10.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.